Restoration of sediment regimes by modifying dam operations

Case study: The Volta Delta, Ghana

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Challenge the future

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by

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This report is the final version of the MSc thesis for the

Master Hydraulic Engineering
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PREFACE

The document that is before you is the final version of my Master thesis. Writing this thesis is the last step in order to obtain my Master's degree in Hydraulic Engineering at the Faculty of Civil Engineering and Geosciences at Delft University of Technology. This report is the product of my research about the effects of modified dam operations on the transport of sediments towards the coast and river bed changes downstream of a dam based on a case study about the Volta Delta in Ghana. The research was a small part of a larger project which is the product of cooperation between the Delft Deltas, Infrastructures & Mobility Initiative (DIMI) and the Delta Alliance Ghana Wing. It was partly carried out at Deltares with the use of its facilities.

Doing this research would not be possible without the help of my committee members, and I wish to take the opportunity to express my sincere gratitude and appreciation for their support during the past few months. I would like to thank Dr. Jeremy Bricker for his time and support since the first day of this thesis process, Dr. Kees Sloff for his technical advise and feedback, Ir. Amgad Omer for his time and extensive explanations to my many questions, Dr. Sanjay Giri for giving the research more depth and Prof. Uijttewaal for his critical and constructive feedback.

Furthermore, I would like to like to thank the next people for their small help at the right moment which helped me make progress at the research: Dr. Peter van Veelen (DIMI) for the opportunity to follow a workshop in Ghana that was financed by DIMI, Dr. Miroslav Marence (Unesco IHE) and Alessandro Cattapan (Unesco IHE) for helping me formulating my research question, Prof. Nick van de Giesen for answering specific questions about the Akosombo Dam, Prof. Kwasi Appeaning Addo (University of Ghana) for explaining me more about the negative impacts of the dam on the delta, Dr. Eric Ofosu (CSIR Ghana) for providing me useful data and Ir. Philip Padi (Volta River Authority) for providing me technical details about the dam and arranging a visit to the dam site.

Last but not least, I would like to thank my family for their mental support and faith in me. It is hard to find words to express my love and appreciation for what they have done for me during the years. A special and heartfelt thanks to my sister Asmaa for her encouragement to start with this thesis topic and her advises during my most important decisions that I always can count on. I wish nothing but the best for her.

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Abstract

The constructing of dams has increased from the second half of the 20th century, and by the year 2000 more than 47.000 dams were built worldwide [1]. Hydropower dams generate approximately 20% of the world's electricity supply, and the contribution of large dams for global food production is about 14% [2].

Unfortunately, as often in cases with human intervention in nature, there are also several negative impacts of dams on natural systems which they are now a permanent artificial part of it. The most severe effect of dams on ecosystems is the alteration of natural flow regimes [3]. In a natural situation where the river transports sediments to the sea, the sediments are deposited near the river mouth where they can be stirred up by the waves and carried along the shoreline by currents. When the incoming sediments are equal to outgoing sediments, the system is in equilibrium, and the coastline is more or less stable. The lack of sediments in the downstream river caused by a dam decreases the inflow of deposits into the sea and thus disturbs the existing equilibrium and leads to coast erosion [4].

Several techniques have been developed to decrease the trapped sediments in reservoirs, e.g., sluicing, flushing and bypassing [5]. The feasibility of each method depends on several (reservoir) characteristics. One of the techniques that this thesis is going to focus on is the choice to modify the discharge through the dam.

Unfortunately, there have not been many studies done yet on such re-operation regimes [6]. This thesis will try to get more understanding of the possible consequences of a modified discharge regime concerning restoration of pre-dam sediment regimes. Because of the many varieties of situations around existing dams, it is hard to come up with general conclusions for all those cases. This thesis treats the case study of the Volta Lake in Ghana. The results of this specific case might be useful for similar situations as a starting point.

In this thesis, the effect of modifying dam operations on both the upstream part of the river (the reservoir) and the downstream part have been investigated. For the upstream section, a simulation is made for a simplified model for a reservoir with similar characteristics as the Volta Lake. According to this model, the modified dam operations are not useful for reservoirs like the Volta Lake. The technique might be useful for shallower reservoirs.

A modified dam operation scenario aims to reintroduce flood pulses to the river. For the downstream part of the river, three aspects of the operation scenarios have been investigated: the discharge peak height during an operation, the duration of such a peak and the extra sediment load that might be brought to the river from the reservoir. The response of the river downstream was measured by two aspects: the sediment load to the ocean and the bed level changes in the river.

To show the response of the river, several simulations were made with different aspects of the operation scenario. At each simulation, only one parameter was changed, e.g., the peak discharge is changed from 1500 to 5000 m^3/s in different simulations. By doing so, graphs could be made to show a correlation between the three operation aspects and the two aspects of the river's behavior.

It turns out that by increasing the discharge, more sediment will be transported to the ocean (with a maximum of 16% of pre-dam sediment load). There will be more non-cohesive material carried to the sea than cohesive material. When there is no extra sediment coming to the river, the river bed will degrade. This will happen at equal rates all along the river which will not change the bed slope.

By increasing the sediment load from upstream of the dam without introducing flood pulses, there will be more sediment transported to the ocean. However, most of the sediment will settle at the river bed causing accretion. The accretion of the river bed will not happen at equal rates along the river so the bed slope will change.

Finally, the change in peak duration will affect neither the transport of sediment nor the river bed. The reason for this might be that the river bed adapts to the different flow conditions in such a way that the same amount of sediments is transported towards the coast.

Contents

List of Figures

LIST OF TABLES

1
Introduction

1.1. Background

The constructing of large dams has increased from the second half of the 20th century, and by the year 2000 more than 47.000 dams were built worldwide [1]. Hydropower dams generate about 20% of the world's electricity supply, and the contribution of large dams for global food production is about 14% [2].

Unfortunately, as often in cases with human intervention in nature, there are also several negative impacts of dams on natural systems which they are now a permanent artificial part of it. The most severe effect of dams on ecosystems is the alteration of natural flow regimes [3]. In a natural situation where the river transports sediments to the sea, the sediments are deposited near the river mouth where they can be stirred up by the waves and carried along the shoreline by currents. When the incoming sediments are equal to outgoing sediments, the system is in equilibrium, and the coastline is more or less stable. The lack of sediments in the downstream river caused by a dam decreases the inflow of deposits into the sea and thus disturbs the existing equilibrium and leads to coast erosion [4].

A reason for the lack of sediments might be the settling in the reservoir created by the dam. In the river part upstream of the dam, the flow velocities start to decrease near the wide entrance of the reservoir, and sediment particles can settle without being held in suspension. The reservoir trap efficiency is defined as the ratio of deposited sediment to the total sediment inflow of a reservoir [17] and depends among others on the particle size, reservoir capacity and mean annual river discharge [18]. The trapped sediments can fill up the reservoir upstream. For hydropower dams, this means a decrease in storage capacity and thus lifetime. The global annual reservoir sedimentation rate is about 0.52% [19] and some reservoirs have been already filled with sediments [5].

Another reason for the lack of sediments might be the elimination of flood peaks. Usually, the yearly discharge pattern of a river shows a peak as a consequence of a rainy season or melting snow upstream. This so-called flood pulse seems to be very important for a river system [20]. Not only for ecological reasons but also for shaping the delta and coast. This is because during a discharge peak more sediment can be eroded and transported downstream. If the new river discharge through the dam is more or less constant and much lower than during the peak, the ability to carry sediments of the river will also be smaller. The absence of a flood pulse can also cause clogging of the river mouth by a sandbar due to the dominance of the strong waves and currents from the ocean compared with the weak river flow [21].

Several techniques have been developed to decrease the trapped sediments in reservoirs, e.g., sluicing, flushing and bypassing [5]. The feasibility of each method depends on several (reservoir) characteristics. Unfortunately, most of those techniques do not seem to be very effective for large reservoirs.

While the techniques mentioned above focus more on the upstream part of the dam, there are also techniques that focus on improving the damaged downstream part of the river. One of such methods that this thesis is going to focus on is the release of discharge pulses through the dam. This technique is not applied much and therefore not much is known about how the discharge pulses interact with the river and what the consequences are of such a method.

1.2. PROBLEM STATEMENT

Construction of a dam can have negative impacts on the river both upstream and downstream of the dam. There are several possible solutions for problems upstream such as flushing and sluicing operations, dredging and bypassing. The effectiveness of these solutions depends among others on reservoir characteristics. When the reservoir becomes larger (capacity-annual inflow ratio >1), these techniques become less feasible as will be shown in section 3.2.

For problems downstream of the dam, one method can be useful: that is modifying the flow that is released from the dam in such a way that it recreates the flood pulse in the river. However, this technique (re-regulating the river flow) is not a commonly used technique. Therefore, little is known about the effects on sediment regimes after modified operations of the dam [6]. Olden et al. (2014) [22] systematically reviewed 113 large-scale flow experiments across 20 countries. In those experiments, the operation regime of a dam was modified in such a way that it would improve the situation downstream of the dam. There was a lot of variety among the experiments (different dam sizes, reservoir capacities, duration of flood pulses and monitored responses). Most of the described experiments focused on relatively small reservoirs with short flood pulses (hours to days) and did not monitor the transport of sediments. None of the listed case studies can be used as a good reference for a situation with a large reservoir, long flood pulse duration (few weeks) and insight in sediment transport.

Thus the problem statement that this thesis aims to tackle is **the gap in knowledge about the influence of long-term modified dam operations on sediment transport in case of a large reservoir**. This will be done based on a case study on the Volta Delta in Ghana.

1.3. OBJECTIVE AND RESEARCH QUESTIONS

The objective of this thesis is: **to understand to what extent modified dam operations can contribute to a restoration of pre-dam sediment regime.**

Modified dam operations mean that the released discharge during a year is not based on energy demand but is changed in such a way as to improve the downstream part of the river. **Pre-dam sediment regime** indicates the transported amount of sediment to the sea and the behavior of the river bed and banks before the dam. **Restoration** can be checked by comparing the new situation downstream of the dam with the old situation before the dam. To obtain this objective, two research questions need to be answered:

1. How do modified dam operations influence the sediment transport in a reservoir?
2. How do discharge pulses influence the dynamics of the river downstream of the dam?

As can be seen from the two questions, the thesis considers the whole river system that is affected by the dam: from the reservoir (entrance) to the river mouth. This, although the considered technique is meant for the downstream part of the river. The idea is to focus on the effects on the river downstream but also have a look at the reservoir.

The first research question focuses on the effectiveness of the applied method concerning reservoir characteristics. This research question aims to find a situation where this technique is the most effective.

The second research question focuses on the effectiveness of the applied method with respect to relevant processes and scenarios such as the discharge peak, its duration and the influence of the sediment load. To answer this research question, several sub-questions need to be answered first. Those sub-questions are:

1. How does the river respond to discharge peaks?
2. How does the peak duration influence the bed's behavior?
3. What is the influence of extra sediment load on the river?

1.4. THESIS OUTLINE

For this thesis, the Volta Delta in Ghana is considered for a case study. Chapter two describes the essential parts of the system. After that chapter three gives a literature review that helps understand the situation and relevant processes playing a role in it. Chapter three is divided into two sections: the first section provides a theoretical background about processes such as sediment transport and section two describes possible existing solutions for problems caused by large dams. Chapter four and five describe the methods that are applied to answer the research questions and also explain the model inputs. Section six shows the results of the simulations, and they are discussed in chapter seven. After that, a conclusion is given in chapter eight.

2

The Volta System

The case study in this thesis is about the Volta Delta in Ghana. The Republic of Ghana is located in West Africa along the Gulf of Guinea near the Equator. The country is bordered by Ivory Coast in the west, Burkina Faso in the north and Togo in the east. The area of the country is about 238.535 km^2 with a coastline of 560 km long. The country has many rivers including the Volta River. One of the most significant events in Ghana's modern history is the construction of the Akosombo Dam on the Volta River in 1965. The dam literally changed the country by creating the Volta Lake upstream of it (figure 2.1 and figure 2.2). In 1982 another dam was constructed downstream of Akosombo which created another small reservoir between the two dams. The next subsections describe essential elements in the system that are relevant to the objective and research questions (see also figure 2.3).

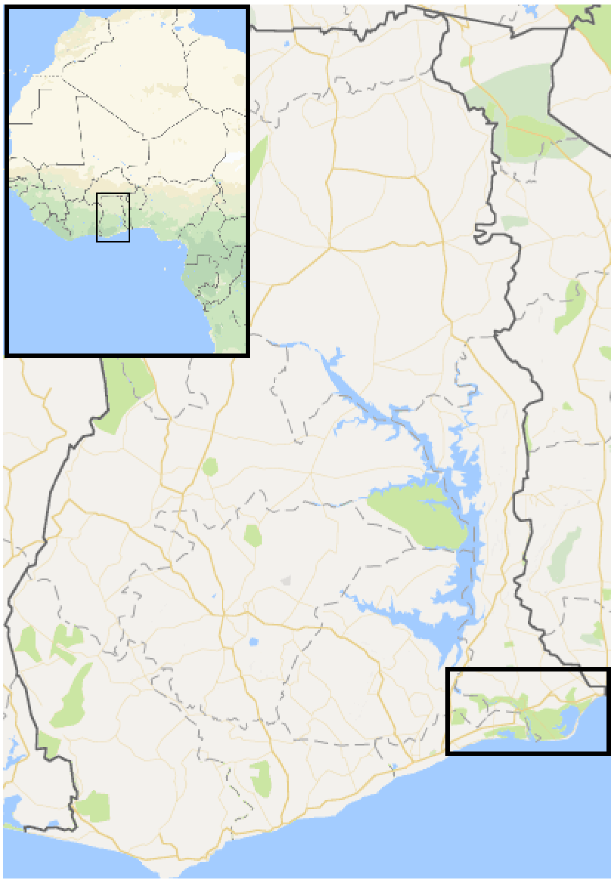


Figure 2.1: A map of Ghana. From Google Maps.

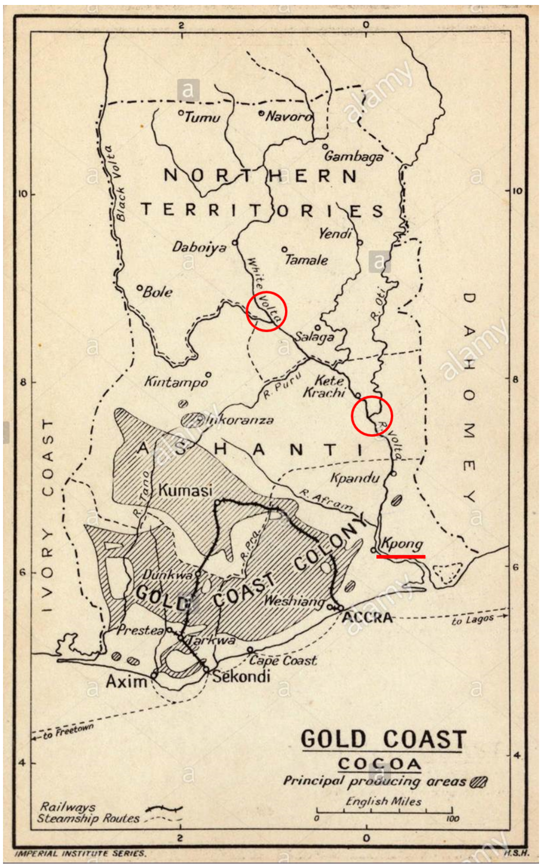


Figure 2.2: A map of Gold Coast (the name of the country before independence). From Alamy Stock Photo.

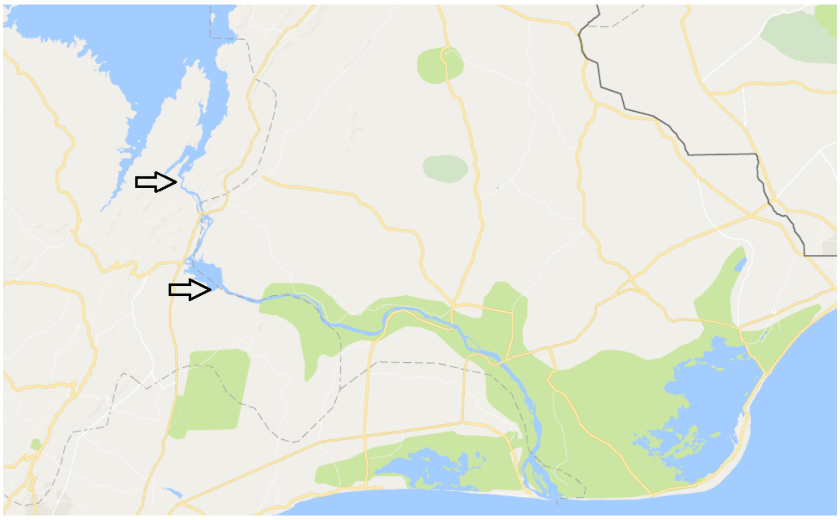


Figure 2.3: The Votla Delta (in green) with the locations of the two dams. From Google Maps.

2.1. TRIBUTARIES OF LAKE VOLTA

The construction of the Akosombo Dam and the creation of Lake Volta (which covers 4% of the total area of Ghana [23]) made a significant and remarkable change on the map of the country as can be seen in figures 2.1 and 2.2.

Before the Akosombo Dam, the White Volta, and Black Volta had a confluence point near Salaga. From the confluence point, they formed the Volta River that flows into the sea. Downstream of the confluence point several rivers flow into the Volta River including the Oti River.

When the reservoir upstream of Akosombo was created, it formed 'arms' near existing rivers. One of those arms stretched out to just upstream of the former confluence point of the Black and White Volta. Therefore those two rivers now flow into the lake separately and together with the Oti River they are the main tributaries of Lake Volta.

The Black Volta is mainly formed by the Sourou River and the Mouhoun River in Burkina Faso. The length of the river is approximately 1.360 km. The mean annual runoff is about $7,7 \times 10^9$ m^3. This forms 18% of the total yearly flows to the Volta Lake [23].

The White Volta also begins to flow in Burkina Faso. The length of the river is about 1.140 km with a mean annual runoff of $9,6 \times 10^9$ m^3. This forms 20% of the total yearly flows to the Volta Lake [23].

The Oti River begins in the Atakora hills of Benin, and it flows through Togo to Ghana. The length of the river is about 940 km. The mean annual runoff is approximately $11,2 \times 10^9$ m^3. This contributes about 25% of the total yearly flows to the Volta Lake [23].

2.2. LAKE VOLTA

By the construction of the Akosombo Dam, a reservoir upstream of the dam has been created. The Volta Lake, which is the name of the reservoir, is the largest man-made reservoir in the world by surface area (8.502 km^2) and the third largest reservoir by volume capacity (148 km^3 by maximal water level) [24]. The average depth of the reservoir is about 18,8 m, and it has a shoreline of approximately 5.500 km. The seasonal variations in the water level in the reservoir are between 2,0 and 6,0m [23].

In the years after the construction of the Akosombo dam, there was no knowledge about the trapped sediments in the reservoir. Freeman (1974) stated that after nine years of completion of the dam, sedimentation rates of the Volta Lake were still not known or even estimated. No effort has been made to monitor reservoir sedimentation. It had no priority for the authorities because sedimentation would not form a treat for the reservoir because of its size [25]. Freeman (1974) plead for more insight in sediment flows into the lake, a bathymetry survey of the lake and information about delta formation in the reservoir.

Between 1980 and 1990 several attempts were made to build a systematic data collection about sediments in Ghana. However, most of the efforts have failed because sampling programs were too expensive to maintain [26]. Akrasi (2005) estimated the amount of suspended sediments yields into the reservoir to be 17×10^6 tonnes per year [26]. This estimation was based on collected data from sampling stations along the main tributaries of the Volta Lake.

2.3. AKOSOMBO DAM

The Akosombo Dam is a rock-fill embankment dam (with clay core) at the Volta River near the small town Akosombo. The dam is 660 m long and 114 m high. It has a base width of 366 m and structural volume of $7,9 \times 10^6$ m^3 [24]. The dam was completed in 1965 and has as main purpose power generation. It is the main supplier of electricity in Ghana. Together with the smaller Kpong Dam downstream of Akosombo, they generate over 50% of the electricity in Ghana [14]. The Akosombo Dam generates electricity using 6 Francis turbines with a total capacity of 1.020 MW. Each turbine is in a penstock with a length varying between 112-116 m, a diameter of 7,2 m and a maximal hydraulic head of 68,88 m. The dam has two spillways with a capacity of 34.000 m^3/s. Each spillway contains six floodgates which are 11,5 m wide and 13,7 m tall each. The Akosombo Dam does not have a bottom outlet. The maximal operating level in the reservoir is 84,73 m, and the minimum operating level is 73,15 m. The water depth at the downstream side of the dam is on average 14,7 m. Figure 2.5 shows a longitudinal section sketch of the dam with the above mentioned values [24].

Figure 2.4: The Akosombo Dam. Author of the picture is unknown.

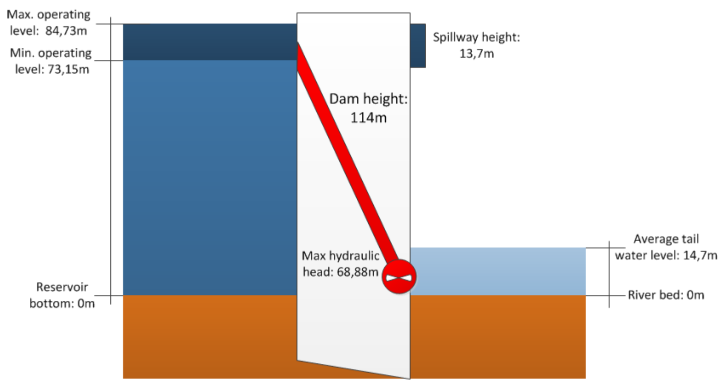


Figure 2.5: A sketch of the dam profile

2.4. KPONG HEAD POND

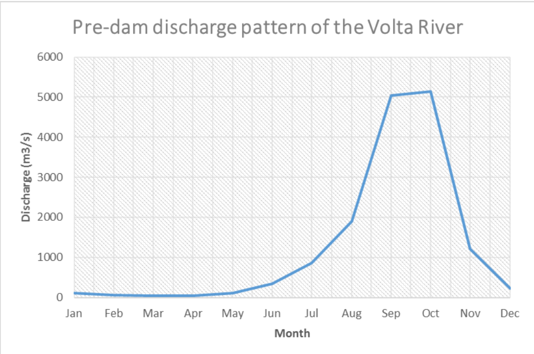
Between 1977 and 1982 the Kpong Dam was constructed approximately 25 km downstream of the Akosombo Dam. This dam, also created a reservoir upstream that is called the Kpong pond. This smaller reservoir has a surface area of 12 km^2 and an average water level of 14,7 m [23]. Because the Kpong Dam operates as a run-of-the-river dam, the water level variations in one year for the reservoir usually are less than a meter (see table C.6, Appendix C). Because of the small size of the lake and the run-of-the-river operation of the dam downstream, it is expected that the reservoir sedimentation is negligible.

2.5. KPONG DAM

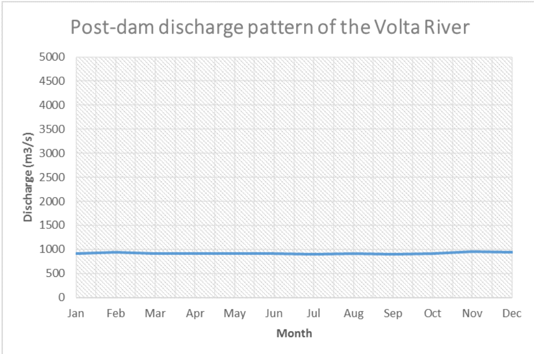
The Kpong Dam is a rock-fill dam near Akuse, a small town, 25 km downstream of Akosombo. The height of the dam is 19 m, and the length is 240 m. The dam was built in 1982 as a run-of-the-river dam. It is used for hydropower generation, irrigation purposes, and municipal water supply. Because of the small size of the reservoir, not much water is stored, and differences between discharges from Akosombo and the Kpong Dam are on average less than 5% (see C.2 and C.3, Appendix C). The design water head for the dam is 11,75 m with four turbines that generate 148 MW together using Francis turbines with a diameter of 8,2 m. The Kpong Dam also has spillways that consist of 15 radial gates, each 11 m wide and 13,5 m high [27].

2.6. THE VOLTA RIVER

Before the construction of the Akosombo Dam, the Volta River was the name for the part from the confluence point of the Black and White Volta to the ocean (figure 2.2). The length of the Volta River before Akosombo was about 450 km[1]. The river discharge was on average 1.255 m^3/s with peak values over 5.000 m^3/s during the wet season (figure 2.6a). The current Volta River, from the Kpong Dam to the sea is about 90 km long. After the construction of the dam and water storage in the reservoir, the discharge through the dam became more or less constant (figure 2.6b).



(a) Monthly averaged discharge of the pre-dam Volta River near Senchi



(b) Monthly averaged discharge of the Akosombo Dam into the Volta River

Figure 2.6: Pre and post-dam discharge pattern of the Volta River. Data obtained from VRA, see C.1, Appendix C

[1] Estimated using Google Maps

2.7. THE COAST

The Ghanaian coast can be divided into three sections [7] with different characteristics (figure 2.7)

- Western coast: starts from the western Ghanaian border to Cape Three Points
- Central coast: starts from Cape Three points to Labadi
- Eastern coast: from Labadi to the eastern boundary of Ghana and can be divided into two subsections:
  - x Western side of the Volta
  - x Eastern side of the Volta: the focus of the thesis will be on this part of the coast (the reason for that will be explained below). This subsection of the coast can again be divided into smaller parts (different studies use a different amount of sections with different borderlines) (figure 2.8):
    - * Section 1: from the river mouth to Wuti
    - * Section 2: from Wuti to Cape St. Paul
    - * Section 3: from Cape St. Paul to Keta
    - * Section 4: from Keta to Kedzi
    - * Section 5: from Kedzi to the eastern Ghanaian border

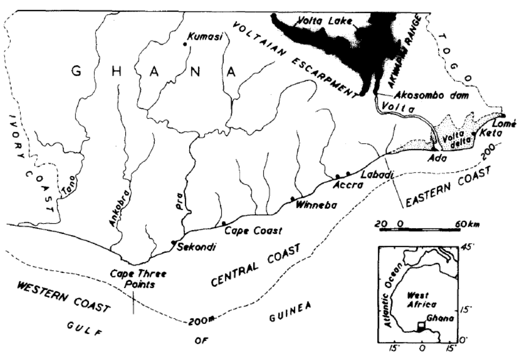


Figure 2.7: The Ghanaian coast. From Ly (1980) [7]

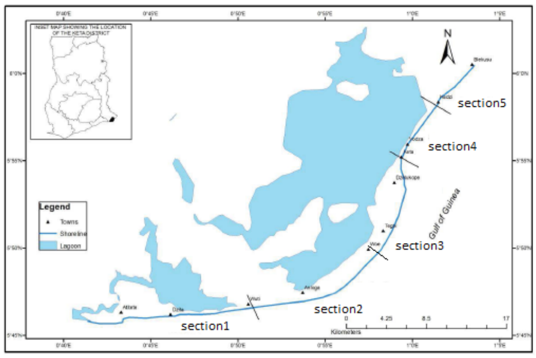


Figure 2.8: The eastern part of the Volta. (Modified) from Appeaning Addo (2015) [8]

The Western coast is characterized by low energetic wave conditions and fine sand. The central coast is characterized by rocky headlands, sandbars, and spits enclosing coastal lagoons. The eastern coast has medium to high energetic wave conditions and coarse sand. It is also part of the Volta Delta [7].

Ly (1980) [7] showed that the western side of the Volta (from Labadi to the river mouth), although being a part of the delta, has never received sediments from the Volta River. This is because of the dominant wave direction (south-west) that causes a longshore drift towards the east.

Although most parts of the Ghanaian coast were not in equilibrium and experienced erosion during the last century, Ly (1980) [7] proved that the Akosombo Dam plays a role in the increase of erosion rates for the eastern side of the Volta only (from the river mouth to the east Ghanaian border). This was done by comparing aerial photos of different locations at the coast before and after the construction of the dam. The study showed that erosion rates at the west side of the Volta River were more or less constant while erosion rates at the east side increased significantly (the erosion rate near Keta between 1923 and 1949 was approximately 4 m/year. Between 1959 and 1975 the erosion rate became 6 m/year and at some places even 8-10 m/year). A summary of the research results of Ly (1980) can be found in figure 2.9.

The research of Ly (1980) is crucial because he compared erosion rates of pre-dam and post-dam situations. However, the results for the eastern side of the Volta were limited to a coastline of about 5,1 km around Keta. Boateng (2012) [9] did research on coastline change for 203 km out of the 540 km Ghanaian coast including the whole eastern side of the Volta by applying Digital Image Processing (DIP) and GIS techniques. Boateng's study showed that there are differences in the erosion rates for the eastern side of the Volta: Sections 1, 2 and 3 experienced an average erosion rate of -0,5 m/year between 1895 and 2002 while sections 4 and 5

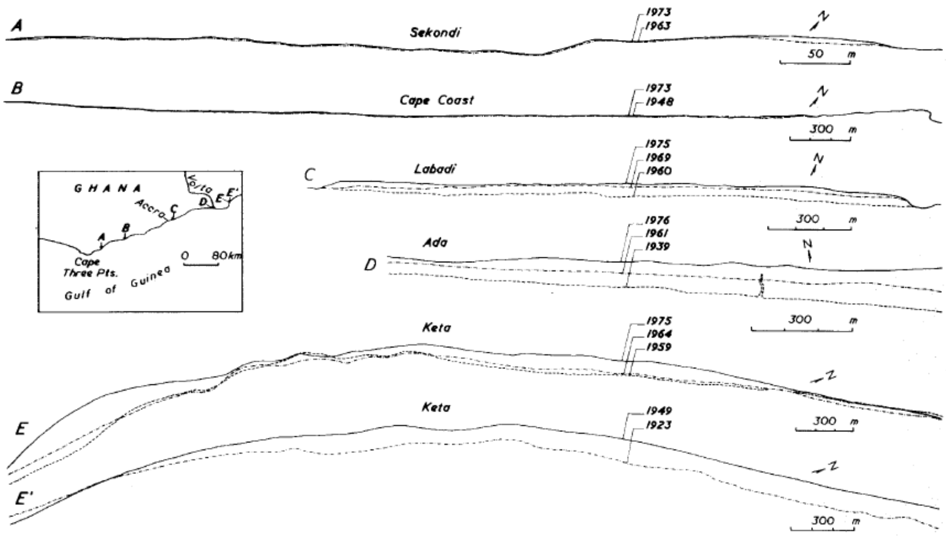


Figure 2.9: Shoreline changes in central and eastern Ghana established from aerial photographs and maps produced between 1923 and 1976. From Ly (1980) [7]

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underwent an average erosion rate of -5,5 m/year in the same period (figure 2.10). Boateng does not give the reason for the differences between the sections. The coastline orientation might explain the difference in erosion rates. In contrast to Ly, Boateng does not make a distinction between pre-dam and post-dam erosion rates for the eastern side of the Volta. Erosion rates after 1965 might be much more significant than erosion rates before. It is possible to compare the results of the two studies by calculating the average value for Ly and comparing it with the average value of Boateng. If we consider the study period of 107 years (between 1896 and 2002), and use the average erosion rate of 4 m/year that was found by Ly for 70 years (1895-1965, 65% of the study period), and 8 m/year for 37 years (35% of the study period), we can find an average of $(0,65 \times 4) + (0,35 \times 8) = 5,4$ m/year for Ly. This result is very close to Boateng's average value. We might assume that Keta is representative for section 3 and 4 meaning that the erosion rates for those sections were approximately 4 m/year before constructing of the dam and about 8 m/year after construction till 2002.

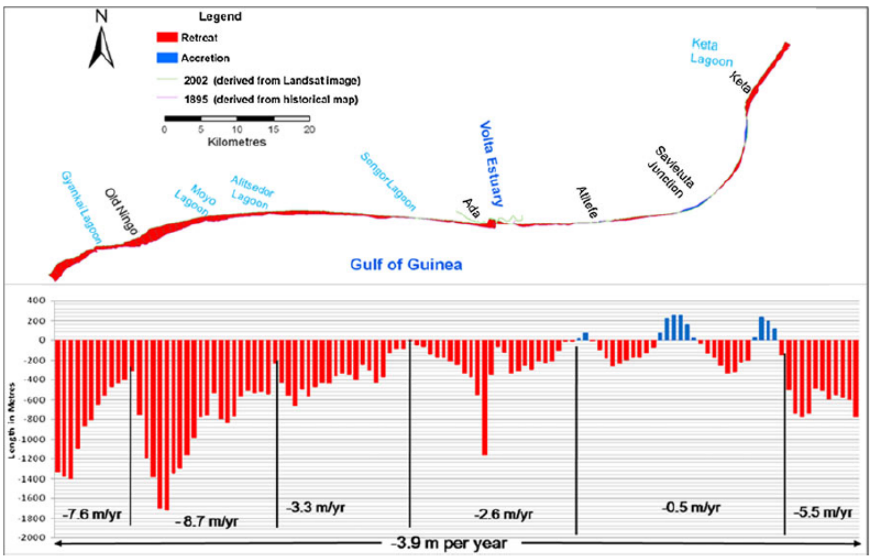

Fig. 7 Coastline change of case study one over the period between 1895 and 2002

Figure 2.10: Shoreline changes in central and eastern Ghana from 1895 to 2002. From Boateng (2012) [9]

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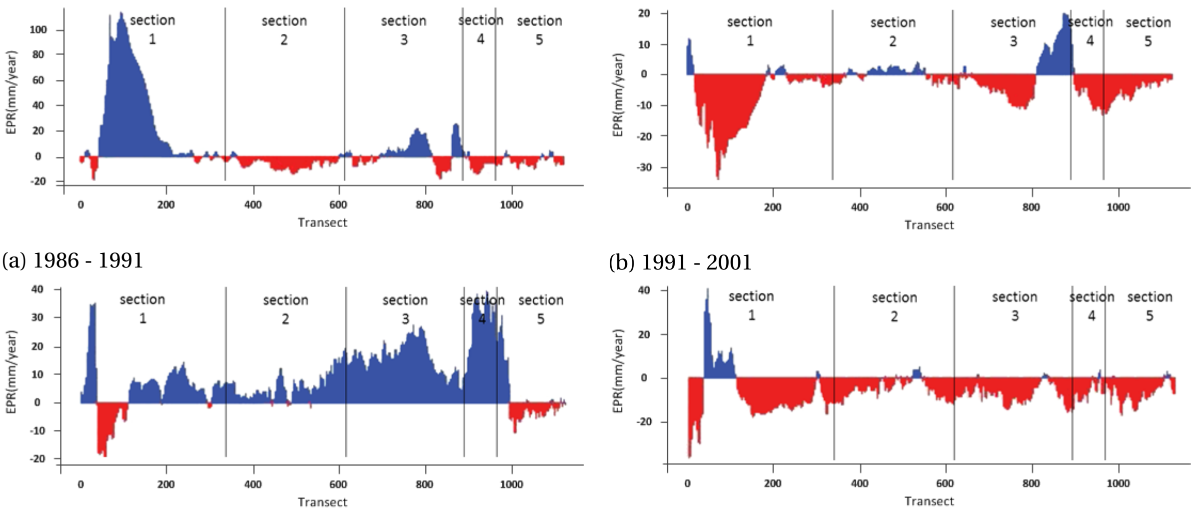
In 2001, the Keta Sea Defence Project (KSDP) was completed. This project consisted of a revetment, six large headland groins, and coast nourishment to prevent coastal erosion near Keta [28]. Appeaning Addo (2015) [8] used satellite imageries from 1986, 1991, 2001, 2007 and 2013 to assess the coastline change for the eastern side of the Volta before and after the KSDP. Figure 2.11 shows the coastline change for the coast during the time intervals. The next observations, possible explanations, and comments can be made:

- for the first two study periods (1986-1991 and 1991-2001), the erosion/ accretion patterns for sections 1, 2 and 3 seem to be exactly the opposite. Areas that experienced erosion in the first study period, experienced accretion in the second period (with different rates) and vice versa. The reason of this might be a cyclic behavior of the coast, e.g., because of a sandbar migration (the time-intervals of this research are more or less the same as the cycle time of a sandbar migration).
- for the first two study periods, the western part of section 1 seems to be very dynamic. This is typical behavior of a river connection with the ocean.
- sections 4 and 5 only experienced erosion in the first two study periods. This might be because of the different shoreline orientation compared with the other sections, or because of the long distance from the river mouth (so less sediment reaches those sections).
- the previous observation indicates that erosion rates over a long time for the first three sections are less than section 4 and 5. This is in line with Ly (1980) [7] and Boateng (2012) [9].
- for the third period of study (2001-2007, after the construction of the sea defenses at section 4), sections 2, 3 and most parts of section 1 experience accretion. Again, the western part of section 1 seems to be very dynamic and hard to understand. Most accretion happened at section 4 (where the groins trapped sediments) and the updrift side of the groins (section 3). It should be noted that not all accretion happened because of natural processes, there was also coast nourishment as part of the project. The average accretion rate for the whole coast was approximately +11,52 m/year
- section 5 did not experience accretion like other sections between 2001 and 2007. This is because the transported sediment along the shore has been trapped at the updrift side, so no sediment reaches the downdrift side at section 5.
- for the 4th period of study (2007-2013), most of the coastline experienced erosion with an average rate of -8,38 m/year. Appeaning Addo assumes that the system at sections 1, 2, 3 and 4 is reaching equilibrium and that the erosion rates will decreases until they reach 0. However, further research is needed to confirm this assumption. The downdrift side of sea defenses will keep eroding.



(a) 1986 - 1991 (b) 1991 - 2001

(c) 2001 - 2007 (d) 2007 - 2013

Figure 2.11: Coastline changes of the eastern side of the Volta between 1986 and 2014. (Modified) from Appeaning Addo (2015) [8].

3

LITERATURE REVIEW

This chapter consists of two sections. The first section deals with theoretical background about sedimentation processes in reservoirs and relevant transport formulas for different types of sediments. Part two describes several sediment management methods that can be applied.

3.1. THEORETICAL BACKGROUND

In subsection 3.1.1 the sedimentation process and delta formation in a reservoir are described. Subsection 3.1.2 describes the different types of sediment (transport methods) and their governing formulas.

3.1.1. THE RESERVOIR

At the entrance of a reservoir, the channel width increases causing the flow velocities and turbulence to decrease. As a consequence of that, sediment particles start to fall to the bottom. The fall velocities depend among others on particle size. Since the suspended sediments consist of different sizes, they will not be equally distributed over the bottom of the reservoir. Figure 3.1 shows essential processes that determine the distribution of deposits in a reservoir. The next paragraphs treat some of these processes.

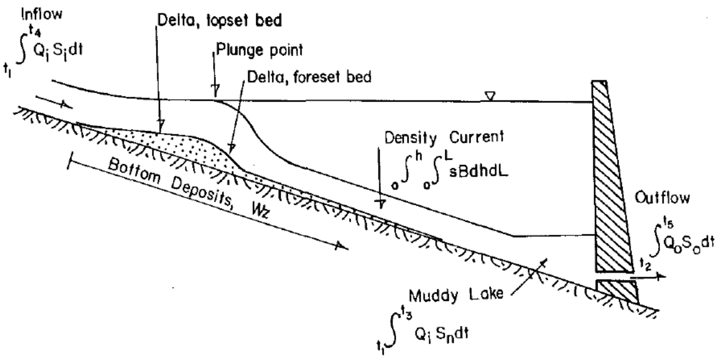


Figure 3.1: Relevant processes that determine sediment behaviour in a reservoir. From Fan (1992) [10]

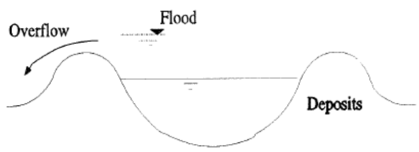


Figure 3.2: Cross section of a reservoir channel with side deposits. From Sloff (1991) [11]

SEDIMENT INFLOW AND DELTA FORMATION

As mentioned before, the depositions in a reservoir are not equally distributed over the bottom. Usually, a delta of similar shape as in figure 3.1 is formed. Two zones in the delta can be recognized: the topset bed and the foreset bed. Since larger particles settle faster, the topset bed contains mainly coarser particles compared to the foreset bed (but finer particles can be present as well). Finer particles can be found on the foreset bed [10]. The topset bed slope is milder than the original river (about 1/2 to 2/3 of the river bed slope) and remains constant during delta progress. The slope of the foreset is about 6,5 times the topset bed slope [11]. According to Vanoni (1977), the behavior of water flow that is entering a reservoir similar to a jet. Because of that, a main channel in the lake is formed with sediment deposits at its sides (figure 3.2).

The deposition distribution can be predicted using either empirical models or mathematical models. Empirical models, however, have limited applicability and can only give a first approximation for the distribution over a few decades [11].

PLUNGING POINTS AND DENSITY CURRENTS

A plunge point marks the transition between the topset bed and foreset bed. As mentioned before this plunge point can be recognized by grain size differences or different slopes nearby the point. At the plunge point, the sediment-laden water might flow under the sediment-free water, and a density current can be formed. When water levels in the reservoir decrease, the main channel will erode, and thus sediments can be transported by density currents. [10]. It should be mentioned that a density current is not always formed in a reservoir. In that case, the sediment transport in a reservoir will behave the same as transport during open channel flow.

3.1.2. SEDIMENT TRANSPORT

To understand and describe the transport behavior of sediment, a distinction between bed-load and suspended load is made. Also, a distinction between cohesive material and non-cohesive material is essential. The next paragraphs describe the governing equations for each type. The first three paragraphs are parts mainly taken from the work of professor Van Rhee (TU Delft) [12]. The complete derivations of the equations can be found in appendix A.

SUSPENDED SEDIMENT: CONCENTRATION PROFILE

The sediment transport is determined with:

$$s = \rho_s \int_{z=0}^{z=H} c(z)u(z)dz \tag{3.1}$$

Where c = volume concentration, u = horizontal velocity of a particle and H = water depth. Hence to determine the suspended sediment transport both the velocity as the concentration distribution must be known. The concentration distribution follows from conservation of mass. Consider a (2D) control volume with length Δx and height Δz. For simplicity let us first consider only transport due to horizontal advection.

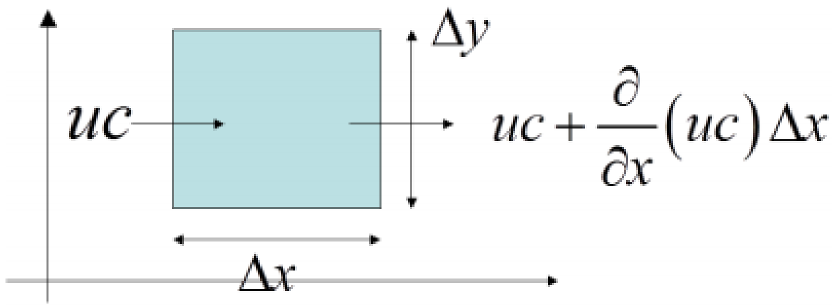


Figure 3.3: Control volume with advection. From Van Rhee (2017) [12].

Transport by advection means that particles are transported by the average velocity in the flow. At the left (West) boundary the volume of particles carried by the flow reads $(uc)_W \Delta t \Delta z$, at the right (East) wall of the control volume the transport reads $(uc)_E \Delta t \Delta z$, where the subscript E or W denotes the West or East wall. The value of the product of velocity and concentration between the east and west boundary can be different, and subsequently, the volume of particles in the control volume will change. the change of volume of particles reads

$$\frac{\partial c}{\partial t} \Delta x \Delta z \Delta t \tag{3.2}$$

The relation between the fluxes at the east and west boundary reads:

$$(uc)_E = (uc)_W + \frac{\partial}{\partial x}(uc)\Delta x \tag{3.3}$$

With this starting point and with further elaboration for stationary uniform flow conditions, this can lead to the next formula:

$$\frac{c}{c_a} = \left(\frac{H-z}{z} \cdot \frac{z_a}{H-z_a} \right)^{\frac{w_s c}{k u_*}} \tag{3.4}$$

Which is the well known Rouse distribution. The exponent in the equation is called the Rouse number and is an important parameter to determine whether the transport mechanism is suspended transport or bed load. When $P = \frac{w_s \sigma}{k u_*}$ the concentration distribution is plotted for different values as a function of depth z/H in figure 3.4. It is clear that for higher values of P, hence for high values of the settling velocity only a limited amount of sediment is suspended in the water column. In literature, a value of $P = 2{,}5$ is often referred to as the limit of suspended transport.

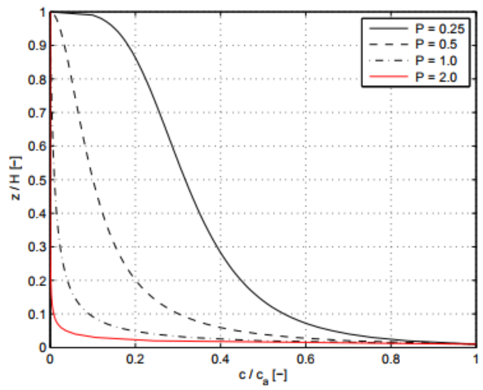


Figure 3.4: Concentration distribution. From Van Rhee (2017) [12].

SUSPENDED SEDIMENT: SETTLING VELOCITY OF NON-COHESIVE MATERIAL

For one-dimensional settling, the particle velocity v_p is computed with:

$$(V_p \rho_s + M_a)\frac{dv_p}{dt} = A_p C_D \frac{1}{2}\rho_w \left|v_w - v_p\right|(v_w - v_p) + V_p g(\rho_s - \rho_w) \tag{3.5}$$

Where V_p = volume of particle, ρ_s = density of particle, ρ_w = density of water, v_p = velocity of particle, v_w = velocity of water surrounding particle, M_a = added mass coefficient, A_p = surface area of particle in flow direction. The added mass is determined with:

$$M_a = C_m \rho_w V_p \tag{3.6}$$

For a stationary situation ($\frac{dv_p}{dt} = 0$) and stagnant flow conditions ($v_w = 0$) this expression reads:

$$v_p = \sqrt{\frac{2\Delta g V_p}{C_D A_p}} \tag{3.7}$$

Where the specific density Δ is defined as:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{3.8}$$

The drag coefficient C_D is a function of the particle Reynolds number, and this leads to different equations for the settling velocity for laminar, turbulent and transition regimes. For the transition regime, often the following equation of Ruby and Zanke is used:

$$w_0 = \frac{10v}{D}\left(\sqrt{1 + \frac{\Delta g d^3}{100 v^2}} - 1\right) \tag{3.9}$$

When a large number of particles is settling in a confined space, the settling velocity of the individual particles is reduced. The influence of the volume concentration on the settling velocity of a monosized mixture (mixture with particles of the same size) is written as [29]:

$$w_s = w_0(1-c)^n \tag{3.10}$$

Where w_0 = the settling velocity of a single particle and c = the volume concentration. A convenient way to compute the value of the value of n is the method of Rowe (1987) [30]:

$$n = \frac{4,7 + 0,41 Re_p^{0,75}}{1 + 0,175 Re_p^{0,75}} \tag{3.11}$$

SUSPENDED SEDIMENT: EROSION AND DEPOSITION FLUXES OF NON-COHESIVE MATERIAL

In a situation where erosion and deposition take place, the sedimentation velocity can be expressed by:

$$v_{sed} = \frac{S - E}{\rho_s(1 - n_0 - c)} \tag{3.12}$$

Where S = sedimentation flux defined as:

$$S = \rho_s w_s c = \rho_s w_0 c(1-c)^n \tag{3.13}$$

And E is the erosion or pick-up flux. It is calculated with empirical expressions and often presented in a dimensionless shape using Φ_p:

$$\Phi_p = \frac{E}{\rho_s \sqrt{g \Delta d}} \tag{3.14}$$

A well-known equation is van Rijn (1984):

$$\Phi_p = 0,00033 D_*^{0.3} \left(\frac{\theta - \theta_{cr}}{\theta_{cr}} \right)^{1.5} \tag{3.15}$$

SUSPENDED SEDIMENT: EROSION AND DEPOSITION FLUXES OF COHESIVE MATERIAL

As for non-cohesive material, the river bed changes depend on the erosion and deposition fluxes. According to the Partheniades-Krone formulations [31], the erosion flux can be written as:

$$E = M \left(\frac{\tau_{cw}}{\tau_{cr,e}} - 1 \right) \tag{3.16}$$

for $\tau_{cw} > \tau_{cr,e}$ where τ_{cw} is the maximum bed shear stress due to current and waves, $\tau_{cr,e}$ is the critical erosion shear stress and M an erosion parameter.

The deposition flux can be written as:

$$D = w_s c_b \left(1 - \frac{\tau_{cw}}{\tau_{cr,d}} \right) \tag{3.17}$$

for $\tau_{cw} < \tau_{cr,d}$ where w_s is the hindered fall velocity, c_b the near bottom concentration, τ_{cw} the maximum bed shear stress due to current and waves and $\tau_{cr,d}$ the critical deposition shear stress. When waves are absent and only the current causes bed shear stress, this can be calculated by:

$$\tau_c = \frac{g p_0 U|U|}{C_{2D}^2} \tag{3.18}$$

where g is the gravitational acceleration, ρ_0 the density of the fluid, U the flow velocity and C_{2D} the 2D-Chezy coefficient.

BEDLOAD TRANSPORT

The bedload transport rate according to Van Rijn (1984) is given by:

$$S_b \begin{cases} 0.053\sqrt{\Delta g D_{50}^3} D_*^{-0.3} T^{2.1} for T < 3.0 \\ 0.1\sqrt{\Delta g D_{50}^3} D_*^{-0.3} T^{1.5} for T \geq 3.0 \end{cases} \tag{3.19}$$

where T is a dimensionless bed shear parameter, written as:

$$T = \frac{\mu_c \tau_{bc} - \tau_{bcr}}{\tau_{bcr}} \tag{3.20}$$

It is normalized with the critical bed shear stress according to Shields (τ_{bcr}), the term $\mu_c \tau_{bc}$ is the effective shear stress. The formulas of the shear stresses are

$$\tau_{bc} = \frac{1}{8} \rho_w f_{cb} q^2 \tag{3.21}$$

$$f_{cb} = \frac{0.24}{\left({}^{10}\log(12h/\zeta_c)\right)^2} \tag{3.22}$$

$$\mu_c = \left(\frac{18\ {}^{10}\log(12h/\zeta_c)}{C_{g,90}} \right)^2 \tag{3.23}$$

where $C_{g,90}$ is the grain related Chezy coefficient

$$C_{g,90} = 18\ {}^{10}\log\left(\frac{12h}{3D_{90}}\right) \tag{3.24}$$

The critical bed shear stress is written according to Shields:

$$\tau_{bcr} = \rho_w \Delta g D_{50} \theta_{cr} \tag{3.25}$$

in which θ_{cr} is the Shields parameter which is a function of the dimensionless particle parameter D_*:

$$D_* = D_{50} \left(\frac{\Delta g}{\nu^2} \right)^{1/3} \tag{3.26}$$

3.2. SEDIMENT TRANSPORT METHODS

This section describes some of the existing sediment management methods and discusses briefly whether they would be useful for large reservoirs such as the Volta Lake.

3.2.1. SLUICING

During sluicing operations, high flows are released through the dam during periods of high inflows to the reservoir. A sluicing operation aims to minimize sedimentation of particles by reducing the retention time in the reservoir and transporting sediment particles as rapid as possible to the downstream side of the dam. The fall velocities of small particles are lower than that of larger particles. Therefore sluicing seems to be more efficient for finer sediments [5].

To decrease the retention time of particles in the reservoir, flow velocities must be increased. This can be done by lowering the reservoir level before river floods from upstream that contain many sediments (figure 3.5). Large flows through the dam can be obtained by outlets with relatively large outflow capacities. For valid results, these outlets need to be at a low position but not at the very bottom of the dam. For dams with small storage volumes, the crest gates can also be used for releasing the increased outflow [5].

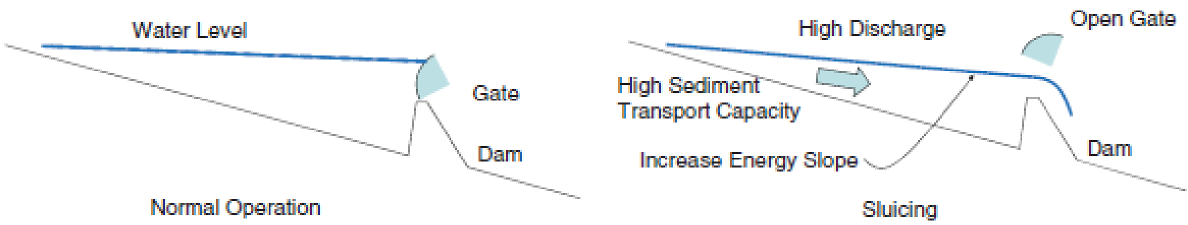


Figure 3.5: Schematic representation of sluicing operations. From Kondolf et al. (2014) [5].

Sluicing operations may be applied to all reservoir sizes. However, it is most useful for narrow reservoirs. The period of keeping the reservoir level low may be different depending on the reservoir size (from hours to a few weeks). For large reservoirs, in order to store clear water and release sediment-rich water, the reservoir level must be kept low during the rising limb of a flood. After that, when the falling limb occurs with lower sediments carried by inflow, the reservoir level can be increased again [32]. Figure 3.6 shows the pattern of increasing and decreasing the reservoir level at the Three Gorges Dam in China. Sluicing operations there seems to be very successful because of the narrow reservoir [5].

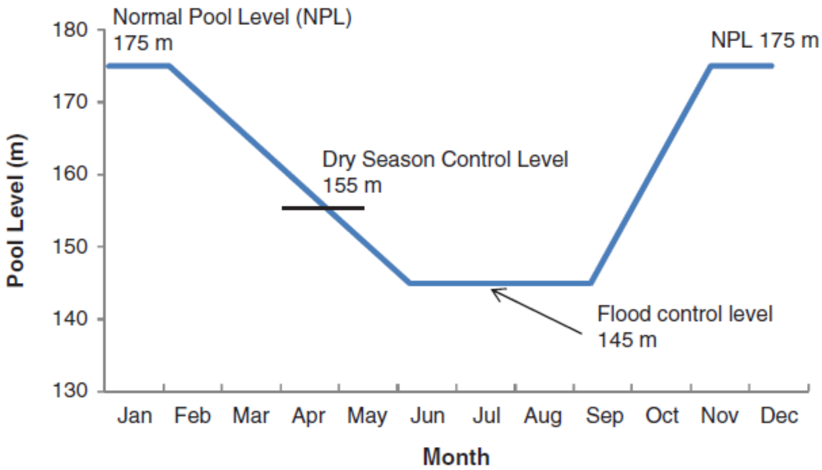


Figure 3.6: Seasonal reservoir operation at Three Gorges Dam. From Kondolf et al. (2014) [5].

3.2.2. SLUICING OPERATIONS IN AKOSOMBO

As mentioned before, sluicing is more efficient for dams with low release gates. Since the Akosombo Dam does not have low positioned gates, the question arises whether releasing sediments through the spillways will be as much as effective as through low located gates. It is possible that higher sediment concentrations are found below the spillway gates and the intake of the penstocks. Therefore less sediment will be transported compared to a situation with low release gates.

3.2.3. FLUSHING

Flushing operations, in contrast to sluicing operations, do not aim to prevent particles from settling in the reservoir. Flushing aims to scour and flush away the already settled sediment particles. For flushing operations, the reservoir needs to be emptied first through low positioned gates. The reservoir levels must be lowered until the flow conditions of the river look like the pre-dam river conditions. This means a free surface flow which allows the erosion of the accumulated sediments upstream of the dam (Figure 3.7) [11].

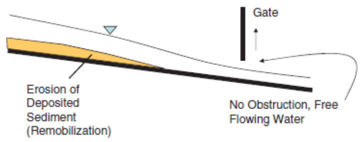


Figure 3.7: Schematic representation of a flushing operation. From Kondolf et al. (2014) [5].

Another difference between flushing and sluicing is that for flushing, in contrast to sluicing, the timing of sediment release might be different from that of sediment inflow during a flood period. It is possible that by flushing the river will be able to transport a large amount of fine sediment during times of relatively low flow. An advantage of flushing during flood periods, however, is that the river will have more erosive energy because of the larger discharges that are available [5].

Several conditions have to be available for a reservoir to increase the efficiency of flushing operations. Those are among others: narrow reservoirs, steep longitudinal slopes, river discharges above the threshold to mobilize and transport sediment and low located release gates [33].

Perhaps the most crucial condition for flushing to be successful is the ratio of reservoir storage capacity to the mean annual runoff (CAP/MAR). According to Sumi (2008), [34] this ratio should not increase 4% for the feasibility of the flushing operation. Figure 3.8 shows some reservoirs around the world with their capacity over the mean annual runoff ratio. It also shows the CAP/MAS ratio which is the reservoir capacity over mean annual inflow sediment. Some successful flushing operations are shown in the plot.

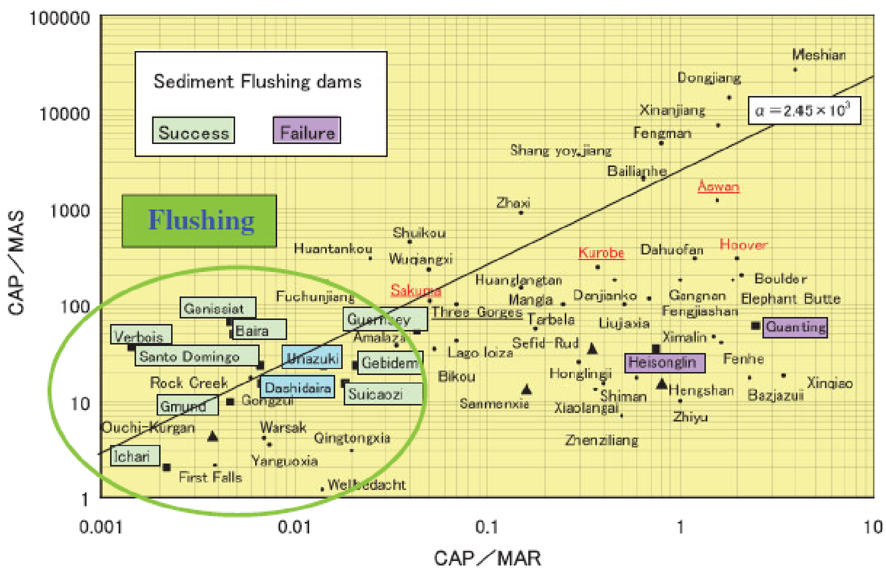


Figure 3.8: Plots of reservoirs with their CAP/MAR and CAP/MAS ratios). From Kondolf et al. (2014) [5].

3.2.4. FLUSHING IN AKOSOMBO

Based on the information mentioned above about conditions that must be provided for a successful operation, it can be concluded that flushing is not the best option for the Volta Lake. The reservoir is not narrow, it is too big (one of the largest man-made reservoir in the world) to empty, it is not steep, and $CAP/MAR > 4$.

3.2.5. MODIFIED DAM OPERATIONS

The aim of modifying dam operations is to reduce the negative effects downstream of the dam. Through some operational changes, e.g., increased minimum flow and periodic releases of flushing flows, dam managers try to restore the natural flow of a river [6]. Because of the variety in dams around the world (size, purpose, impacts, etc.) and because of the different characteristics of river systems, it is not possible to come up with one standard example of a modified operation that fits all the situations. Richter and Thomas (2007) [13] describe an assessment framework that can be used to help determine the best re-operation regime of a dam (figure 3.9). The first step of this framework is to assess the dam's effect on the river flow regime and to understand the nature and magnitude of the hydrologic changes caused by the dam. The next step is to describe the ecological and social consequences of those hydrologic changes. Once that is done, the goals of intended dam re-operations can be specified. After that, the design and implementation of the re-operation strategies can be done. The last step is the assessment of results against goals. If we consider the Volta Delta and possible re-operation strategies of the Akosombo and Kpong dams, it can be said that several studies have already been done on the first three steps.

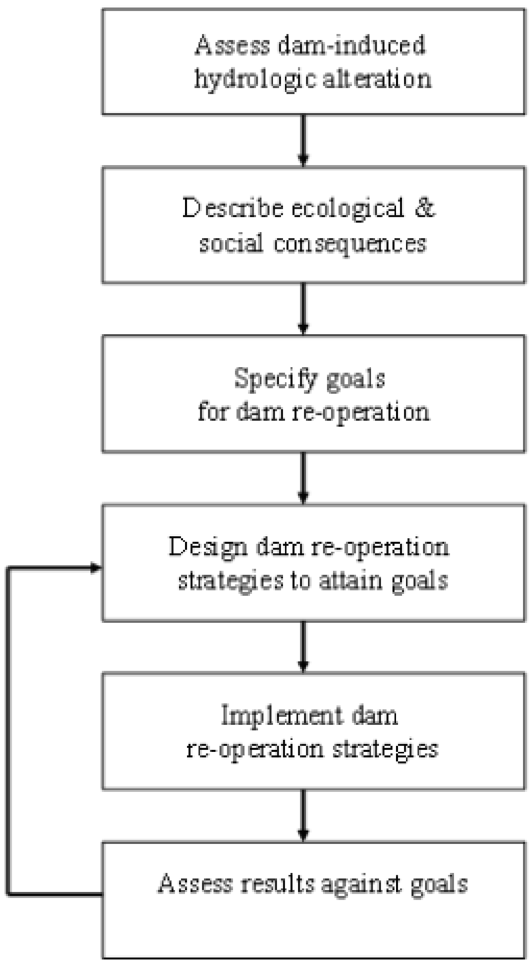


Figure 3.9: Steps of a framework to help determine a re-operation strategy. From Richter and Thomas (2007) [13].

3.2.6. OPERATION STRATEGIES FOR THE AKOSOMBO DAM: EXPLAINING THE CONCEPT

The next step in the above-mentioned framework is to design dam re-operation strategies. This paragraph describes four design strategies that were made for the Akosombo Dam and explains the possible consequences of these strategies on power generation briefly. The next section gives a summary of a feasibility study and describes the effects of the re-operation strategies in more details.

When conventionally operating a hydropower dam, the inflow of the reservoir upstream is not constant in time because of the seasonal variations of the river discharge upstream of the reservoir. The outflow of the reservoir through the dam, however, depends mainly on the purpose of the dam. For a hydropower dam, this is power generation, and thus the reservoir outflow through the dam depends on the energy demand. For Ghana, this energy demand is more or less constant during a year. Since the generated power is linearly proportional to the discharge (and hydraulic head) according to $P = \epsilon\rho g H Q$, the outflow through the dam will also be more or less constant. Because of the differences in inflow and outflow, the reservoir water level will fluctuate (which will cause a small decrease in the produced power). Figure 3.10 A-D summaries this.

For the reoperated dam, this works differently. The aim of the concept is to vary the outflow through the dam, not based on energy demand. In the specific case where the inflow equals the outflow, the water level in the reservoir will be constant. Because of that, the potential power that can be generated will have the same shape as that of the outflow discharge (see figure 3.10 E-H).

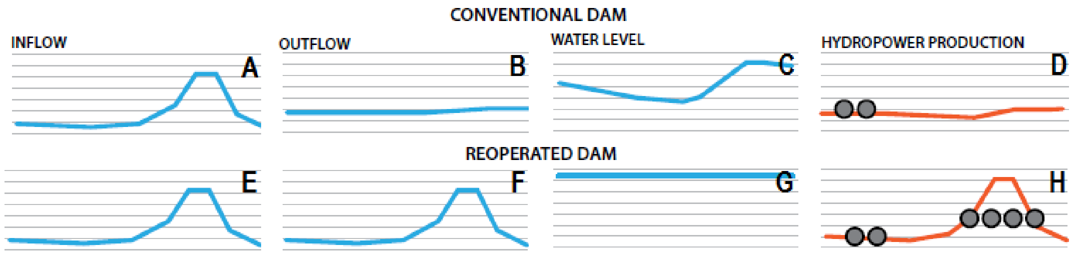


Figure 3.10: Influence of reservoir inflow and outflow on power generation of dams. From Mul (2017) [14]

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The previous figure is from a feasibility study for re-operation strategies for the Akosombo Dam. In subfigure H, two black dots are sketched which indicates that usually two of the available six turbines are capable of generating the power that is needed. When re-operating strategies are applied, the two turbines will reach their capacity, and the other four can be used as well. If the outflow through Akosombo exceeds a certain value, the capacity of the combined turbines will also be reached, and therefore not all potential power can be generated because of the capacity limit as will be shown later. Figure 3.11 shows the hydrographs of four re-operation scenarios that have been developed for the Volta Delta based on different considerations.

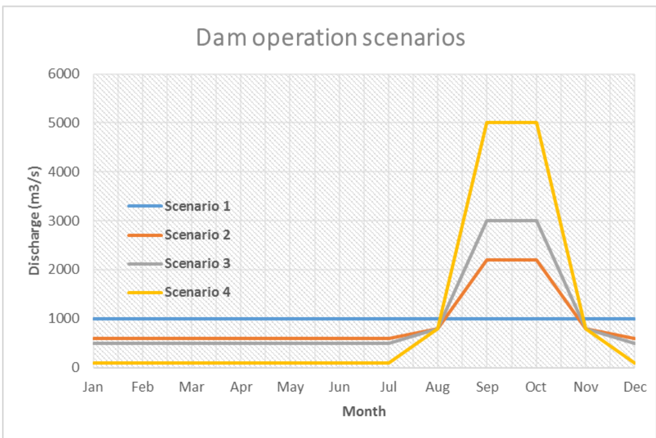

Figure 3.11: Restoration hydrograph scenarios. From Mul (2017) [14]

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3.2.7. OPERATION STRATEGIES FOR THE AKOSOMBO DAM: IMPACT ON POWER GENERATION

To get insight into the effects of the re-operating strategies on power generation, a small calculation is made. As mentioned before, the potential power generation can be calculated using $P = \epsilon \rho g H Q$, where ϵ is an efficiency coefficient, ρ is the water density, g is the gravitational acceleration, H is the hydraulic head and Q the discharge.

The average variation in water level at the reservoir is less than two meters (see table C.4 appendix C) and compared with the total hydraulic head and discharge (variation) it can be neglected in the estimation of generated power. When all values are known except for the efficiency coefficient, the theoretical value for P for all scenarios can be calculated. It is possible to find the efficiency coefficient ϵ by comparing the averaged theoretical value for power with the actual generated averaged value which is known from records of the Volta River Authority (see table C.5 appendix C). By doing so, an efficiency coefficient of 0,67 can be found, and thus the potential power can be plotted. Figure 3.12 shows the potential power (in MW) that can be generated during a year for different scenarios. As mentioned in the previous paragraph, the power generation will be limited by the capacity of the turbines which is 1020MW for the Akosombo Dam. This means that some extra discharge of scenario 3 and 4 will be lost without using it for power generation. Another important aspect is the minimum power that needs to be guaranteed. According to Mul (2017) [14], 6 GWh/ day is required which corresponds with a power of 250 GW. This means that (in this simplified situation), only scenario 2 will be technically possible.

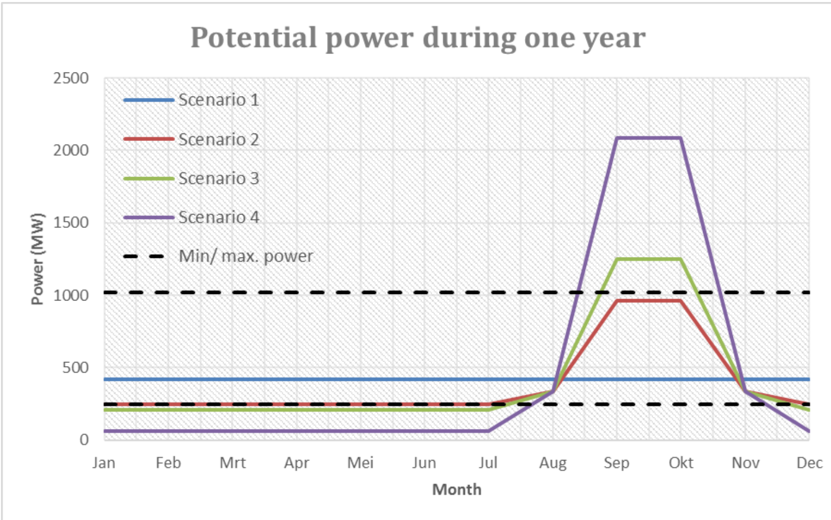

Figure 3.12: Potential power during a year for different scenarios using the hydrographs from Mul (2017) [14]

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It should be mentioned that the actual runoff of the river is higher than that is assumed for the construction of the above-mentioned scenarios. In the constructed scenarios, a runoff that corresponds with a constant yearly discharge of 1000 m^3/s is used and redistributed in such a way to create scenarios 2, 3 and 4. However, according to discharge records (see table C.1 appendix C), the average discharge of the Volta River is 1255 m^3/s. When that is redistributed to create scenarios with the same peak as scenarios 2, 3 and 4, higher discharges during dry months can be found. Figure 3.13 shows the potential power in a year when these values are used.

It can be seen that scenario 3 now can generate more power than the minimum required. This means that the only limitation for this scenario is the capacity of the turbines. With six turbines combined, a total capacity of 1020 MW can be generated. This means that three or four extra turbines are needed. Scenario 4 seems to be impossible to apply since it generates power lower than that is required most of the time.

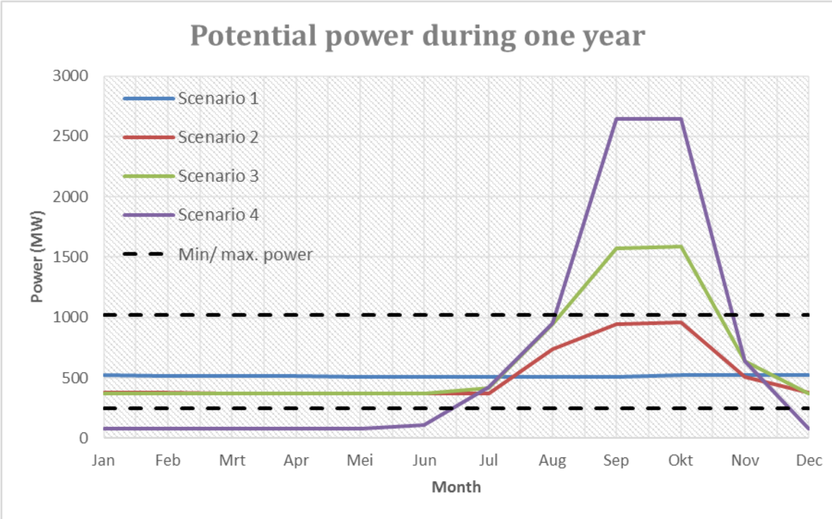

Figure 3.13: Potential power during a year for different scenarios using hydrographs based on runoff records

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3.2.8. OPERATION STRATEGIES FOR THE AKOSOMBO DAM: THE FEASIBILITY ASSESSMENT

The feasibility assessment of the possible scenarios in the above-mentioned study was not only depending on improvements of the environmental conditions, but also technical requirements, economic, and social feasibility. With the current characteristics of the dam, no extra power can be generated when the spillways are in use during the wet seasons. Therefore the spillways need to be equipped with additional turbines to generate power when they are used. During dry seasons, the reduction in electricity generation needs to be compensated by other ways of power generation. Without tackling those issues, the power generation of the dam will decrease when the chosen scenario is closer to the natural river pattern (and further away from the current regime).

For the economic feasibility, a cost-benefit analysis has been conducted for the different scenarios. It turns out that the benefits for the downstream communities (who suffer the most from the negative effects of the dams) will always be smaller than the cost of reduced power generation. Hydropower generates the highest income and contributes to 60-78% of the total benefits. Therefore, as from an economic point of view, the best scenario seems to be the current operation regime.

For the social feasibility, it turns out that the scenario with the highest total benefits (the current regime) is the least beneficial for the downstream communities. However, those communities are interested in addressing the issues regardless of the proposed solution approach. Therefore dam reoperations do not seem to be necessary.

It should be noted that this study did not focus on sediment transport from the Volta Lake during the re-operation scenarios. Apparently, because of the generation of more than 50% of the national power demand, any small changes in the current operation regime would not be beneficial. Studies on details of the system like sediment transport do not seem to be needed at the moment. Despite this fact, research questions like the ones this thesis is focusing on are not useless. The insight into the behavior of sediments in a large reservoir during modified dam operations might be useful in the future (in other places outside Ghana).

4

METHOD & MODEL INPUT: RESERVOIR PART

The first research question of this thesis is **"How do modified dam operations influence the sediment transport in a reservoir?"**. This chapter describes the method that is used that deals with that question about which only focuses on the reservoir part. As said before the described method of modifying the dam operation is mainly meant for the downstream part of the dam. However, it is interesting to see whether this method also influences the upstream part and if that is the case, how this is done.

As a starting point, a model for the Volta Lake needs to be constructed. However, since there is no bathymetry available for the lake, it is hard to start with a model for it. Therefore, a simple schematized model with the same characteristics as the Volta Lake is made. Section 4.1 describes how that model is constructed. In section 4.2 it is shown how the model response compared with known data about the real lake and finally, section 4.3 describes a set of simulations that are done to investigate the effect of the method on the reservoir.

4.1. CONSTRUCTION OF A SIMPLE MODEL

With the known volume and surface area of the lake, a simplified model is constructed in such a way that both values correspond with the real values of the lake. The shape of the lake in the model is a rectangle. Figure 4.1 shows a part of the grid that is used for computations. It can be seen that the grid contains only three cell columns with the middle one much smaller than the other two. The middle column of cells has the same width as the average width of the Volta River. The idea is that the simplified lake is the deepest in the middle because there used to be the old river channel before construction of the dam. From the middle of the lake, the bed gradually decreases towards the lakeside. Figure 4.2 shows a cross-sectional shape of the lake. The bed also has a slope along the lake.

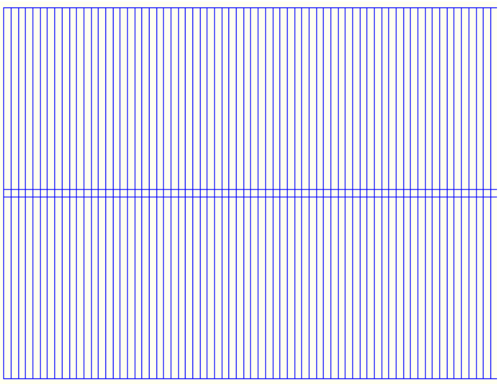

Figure 4.1: The computational grid for the simplified lake. The flow direction is from left to right.

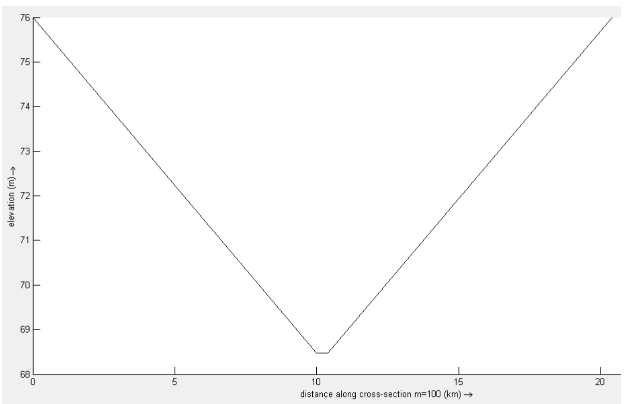

Figure 4.2: Cross-section of the simplified lake

4.2. VALIDATION OF THE MODEL

Usually, by validation of a model, one can check whether the constructed model is representative for the case that he or she wants to investigate. Since for the reservoir case, the constructed reservoir does not look like the real one, the word 'validation' might not be precisely correct. In this case, the simplified model for the lake does not aim to mimic the exact behavior for the Volta Lake, but it may help understand whether modified dam operations influence a large reservoir with similar characteristics. It is said before that the volume and surface area of the constructed lake is in the same order of magnitude as the real reservoir. By comparing a specific behavior from the simulation from the model with a known response, one can check whether the model is representative of a reservoir with the same characteristics as the one in Ghana. If that is the case, the results can be used as a starting point for investigating the effect of modified operations on such reservoir although the exact behavior is not simulated.

A known behavior that can be checked is the fluctuation of the water level in the reservoir. Table C.4 in appendix C contains all monthly averaged water levels at the Volta Lake since the construction of the dam until 2012. Figure 4.3 shows the averaged value for each month during a year. Figure 4.4 shows the water level fluctuation in the reservoir from the simulation. It can be seen that it is in the same order of magnitude. Based on this, it can be said that despite that simulated results are not exactly that what is happening at the Volta Lake, the behavior will be similar. Not only for the Ghanaian case but also for other large reservoirs with similar characteristics.

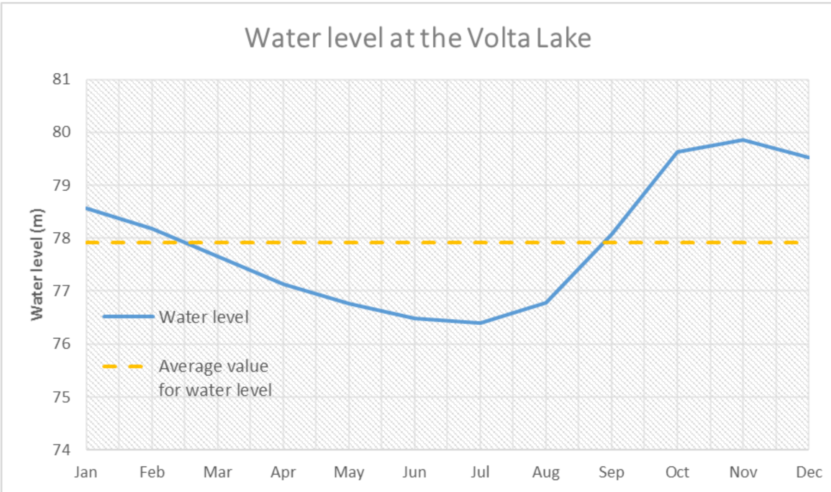

Figure 4.3: Water levels at the Volta Lake

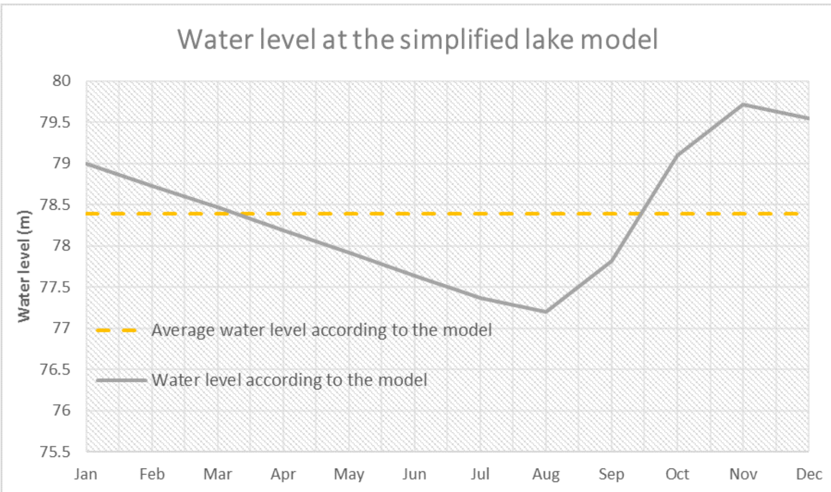

Figure 4.4: Water levels at the simplified lake model

4.3. SIMULATION SCHEMES

To check whether the new dam operations will affect the situation in the reservoir, two simulations for the reservoir are made. The difference between the two simulations will be the discharge from the dam downstream of the reservoir. In the first simulation, the discharge through the dam will be kept constant (1000 m^3/s) like the current regime of the dam at the moment. In the second simulation, the outgoing discharge through the dam will be the same as the incoming discharge from the river into the reservoir (figure 4.5).

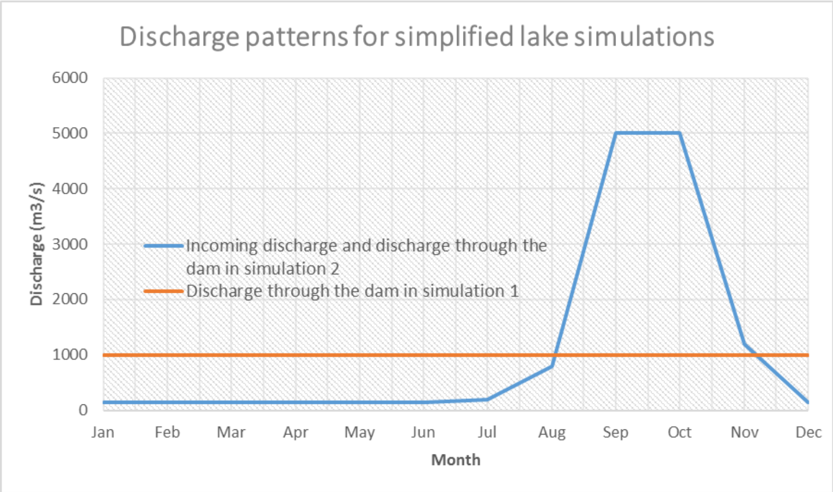

Figure 4.5: Discharge patterns for simulation 1 and 2

After these simulations, another two simulation sets (each including two simulations again) are made: these simulation sets aim to check whether changing essential reservoir characteristics will influence the results. The two reservoir characteristics that are changed during these simulations are the depth and width of the reservoir.

5

METHOD & MODEL INPUT: RIVER PART

5.1. INTRODUCTION

The second research question of this thesis is **"How do discharge pulses influence the dynamics of the river downstream of the dam?"**. In order to answer this question about how different dam operations can influence the morphology of the river, first, the effects of the various operations need to be known. This can be done using model simulations. The idea is to construct a model and make simulations for different operation scenarios. The results of these simulations will show what the different effects are of different scenarios. However, the question "why this is happening" will not be answered yet. This is only a first step.

The next step to answer the question is to look for a correlation between the considered aspect and the expected behavior of the river according to the model. This can be done by varying one parameter during the simulations. So in case of considering the effects of the discharge peak on the bed, simulations with different peak heights will be made. It is expected that by comparing the results of these simulations, a correlation can be found between the cause and the consequence. This is the second step.

The last step to answer the research questions is to try to explain the observed correlation and behavior. This explanation can be tested by making simulations again and check whether the obtained new results match with what was expected based on the explanation.

Before using the simulations to answer the research questions, the constructed model needs to be reliable. A reliable model means that at least the qualitative behavior of the river can be predicted when used for other situations with different input parameters. Since the case study of this thesis is about Ghana, the Volta River will be simulated. The question is whether the constructed model using available input information about the situation will also produce results that predict not only the qualitative behavior but also the quantitative behavior of the river. This can be checked by validation of the model.

So the idea is to use the available data and information about the Volta Delta to get results that not only answers the research question but also gives insight into that specific case. If this is not possible, then the extent of uncertainties should be made clear in order to say whether this model is reliable or not and what the limits are of the model.

The following list sums up all the steps that need to be done:

- Constructing and calibration of a hydrodynamic model
- Constructing and calibration of a morphodynamic model
- Investigating the influence of the discharge peak height
- Investigating the influence of the discharge peak duration
- Investigating the influence of the sediment load
- Setting up a method to compare the results of different scenarios

In section 5.2 the construction of the hydrodynamic part of the model is described. This part of the model is mainly to check whether the used river profile in the model is representative for the Volta River.

In section 5.3 morphology is added to the model. By comparing the model results with known information about the river, validation can be done.

Section 5.4 describes other relevant aspects for the model. In section 5.5 it is explained how the model is validated and finally, section 5.6 shows the scheme for the needed simulation sets.

5.2. CONSTRUCTION AND CALIBRATION OF A HYDRODYNAMIC MODEL

As mentioned before in the previous section, a model needs to be set up to show what the consequences are of different operation scenarios. Since the case study is about the Volta Delta, a model will be set up using inputs for that case. First, it is tried to construct a model that is representative of the Volta Delta both qualitatively and quantitatively. Due to the lack of data as will be shown later, it will be hard to construct a model that is 100% accurate in a quantitative way for the Volta Delta. Because of that, some assumptions are necessary during the set up of the model.

First, a hydrodynamic model for the Volta River is constructed. The hydrodynamic model aims to check whether the river bed profile used in the model is correct or not. Once that is done, morphology will be added to the model to check other inputs like sediment characteristics. Usually, calibration is done by comparing results like water levels from the model with known water levels along the river. However, this is not possible for the Volta River since there are no water level measurements available along the river. An alternative for that could be comparing the water level results with an outcome of a different study that has been done for the Volta River. In that case, one should know to what extent the used results are reliable as a reference point.

Below, a description of such a study is given. First, a summary of the study is provided including the aim of it and the obtained results. After that, some critical remarks are made about the way the model in that study is constructed. Finally, it is explained how this study can be used for the calibration and whether calibration via this way is reliable or not.

5.2.1. THE CSIR STUDY: SUMMARY AND RESULTS

In 2017, the CSIR Water Resources Institute in Ghana published a paper with the name "Floodplain hydrodynamic modeling of the Lower Volta River in Ghana." In this thesis, it will be simply called "the CSIR study." This study aimed to examine the hydrodynamic effects (mainly the water level changes) of different dam operation scenarios on the area along the Volta River downstream of the dams (based on the four scenarios as described by Mul (2017) To do so, data about the river bathymetry was collected during field measurements. During the measurements, sediment data was also collected. The collected data about sediment concentrations were only used to estimate the suspended sediment load based on a relation found in the literature. It was not used for the model since morphology was not included.

The specific objectives of this study as described in the paper were:

1. To acquire river cross-sectional, floodplain geometry and fluvial sediment data of the lower Volta River by field activities
2. To determine the levels of floodplain inundation resulting from the reoperation dam release scenarios through hydraulic and hydrodynamic modeling of the river and its floodplains
3. To estimate suspended sediment load in the river

For river cross-sections and bed profiling, the Acoustic Doppler Current Profiler (ADCP) instrument was used. The device was rigidly attached in a vertical position on a boat and data about the river depth were collected while the boat is moving between the river banks. This was done for 60 spots, and thus 60 cross-section profiles were collected. For the longitudinal profile, the ADCP was used again while moving along the middle of the river from Akuse towards the ocean.

For the floodplains, information about their extent and elevation were collected. For the elevation, the instruments "Trimble GeoXH Explorer 6000 series deferential GPS" and/or the handheld "GARMIN GPSmap 60Cx" with an accuracy level of ±100 mm and ±3000 mm, respectively were used. According to the report, eight reference points with known elevations along the river were used as control points to check whether the GPS equipment was functioning correctly during the study. By combining the data about the cross-sections, longitudinal profile and floodplains, a set of geometric data of the Volta River were obtained.

As for the last field activity, sediment samples were collected at ten sections along the river for analyses about (suspended) load transport and composition of the bed. This was done by a bed-load grabber for bed-load and an integrated sampler for suspended sediment. With the obtained suspended load concentrations, the sediment discharge could be estimated using the relation $Q_s = 0,0864 C_s Q_w$.

Here Q_s is the total suspended sediment discharge in tonnes/day, Q_w is the river discharge in m^3/s and C_s is the total suspended sediment concentration in mg/l (Ofori et. al (2015) [35]). Figure 5.1 shows the downstream part of the Volta River with measurement locations.

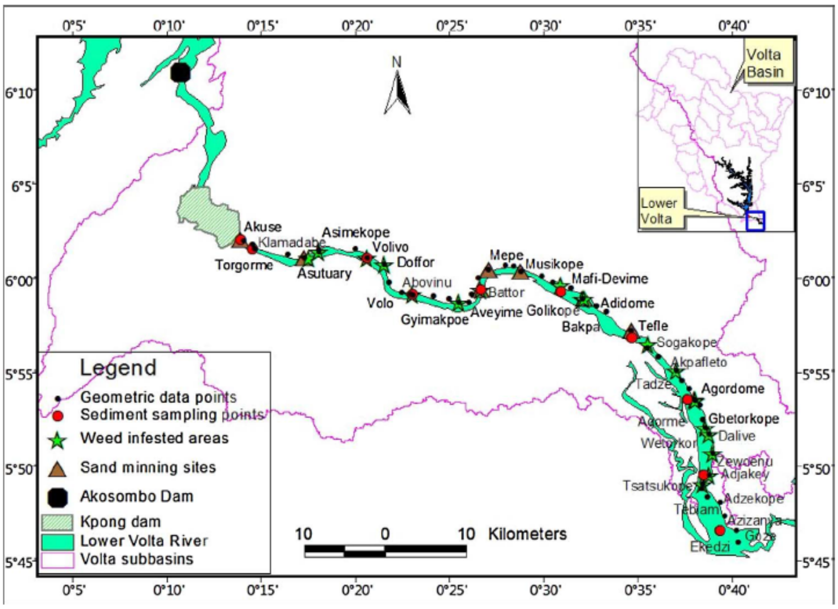

Figure 5.1: Longitudinal river reach showing locations where data were collected on the lower Volta River. From Logah et al.(2017) [15].

With this collected data, a one-dimensional HEC-RAS model was set up to investigate the impact of increased discharge from Akosombo. For the upstream boundary condition, the discharge from the dam was used, and for the downstream boundary condition, the normal depth of the river was used. The Manning's parameters for channel resistance that were used are 0,06 and 0,033 for the banks and main channel respectively. Those data are based on literature (Fischenich (2000) [36], Phillips and Tadayon (2006) [37] and Khayyun (2008) [38]).

The results from the model can be seen in the next figures. The bed level in the middle of the channel, the bed banks and the expected water level for different discharges are modeled are plotted.

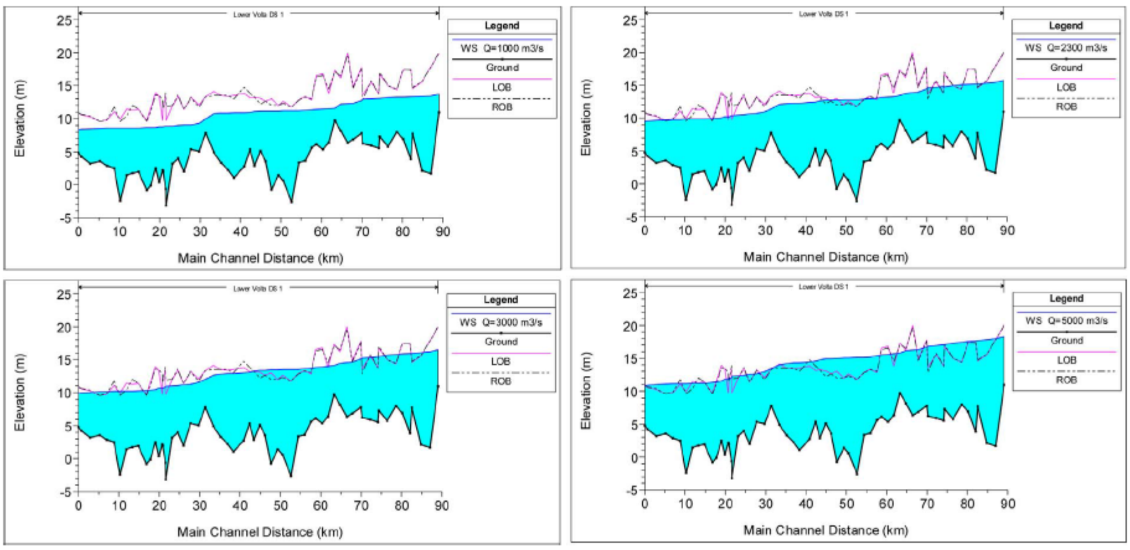

Figure 5.2: Longitudinal surface water profile of the Volta River using peak flow from reoperation scenarios. From Logah et al.(2017) [15].

5.2.2. THE CSIR STUDY: CRITICAL REMARKS ON THE HEC-RAS MODEL

Before using the CSIR study and the HEC-RAS model data for a Delft3D model, some side notes about the Ghanaian HEC-RAS model need to be mentioned:

In the CSIR report, the normal depth is used as the downstream boundary condition. The reason here for is that it is a commonly used condition for a HEC-RAS model. In the figures of the plotted water levels, it can be seen that the water level downstream (near the ocean) is varying between 8,5 and 11m for a discharge ranging between $1000m^3/s$ and $5000m^3/s$ respectively. This sounds odd since the downstream boundary is very close to the ocean and not a point far upstream of it. It would be better if the downstream boundary is a constant value (in the absence of tides) for all different values for the discharge. This constant value is the mean sea level.

When trying to find the mean sea level elevation value, another issue with the CSIR report becomes clear, that is that the mean sea level is not at an elevation of 0 meters. Near the mouth, the bed level has an elevation of 5 m according to figure 5.2. Also, the bank elevation at the mouth is 11 m which seem to be very high for a delta area (if the elevations were measured with the mean sea level elevation as a reference point (0 m)).

Although there are unclear details about the (reference point for) elevations, this model is not useless as a reference for the Delft3D model. The reason here for is that the river profile and bank profiles are measured correctly. The only thing that is missing is the reference point for elevation which does not seem to be the mean sea level. Thus the long profile of the river only needs to be 'shifted down' to match the mean sea level as reference points. To do this, elevation maps of Ghana were used to check the bank elevations upstream of and downstream of the river.

Elevation data from the Shuttle Radar Topography Mission (SRTM) was used with QGIS to check the elevations and to find out what the elevation is of the mean sea level used in the CSIR study. It turned out that used values for elevations in the CSIR model are approximately 9 m higher than when the elevation values with the mean sea level as a reference point.

By doing this, it can be concluded that the water depth near the mouth is about 4 m. When comparing this with the results from the HEC-RAS model for a discharge of $1000m^3/s$, it can be seen that the situation is more or less the same (the depth in the HEC-RAS model is about 3,5m). So apparently the water level with this simulation matches the mean sea level and therefore well represents the current situation along the river.

For the Delft3D simulation, the mean sea level at the elevation of 9m will be used (since that is easier to compare with the HEC-RAS results). The shift in reference point does not change anything about the (physical) effects. The only thing that will be different between the Ghanaian HEC-RAS model and the Delft3D one is that instead of using the normal depth as a boundary condition, the water level at sea will be used.

5.2.3. DELFT3D: CALIBRATION OF THE MODEL

To make a similar model in Delft3D, the same inputs of the HEC-RAS model can be used. However, the data for the 60 cross-section profiles are not available. Therefore the cross-section profile shape should be assumed. With the known elevations of the bed and bank levels, a trapezoidal shape can be assumed. Three different values for the width of the lower side are used (See figure 5.3). Three simulations are made with the three different shapes in Delft3D, and the obtained results for the water levels are compared with the results from the HEC-RAS model to check which form is the most representative for the cross-sectional shape of the river.

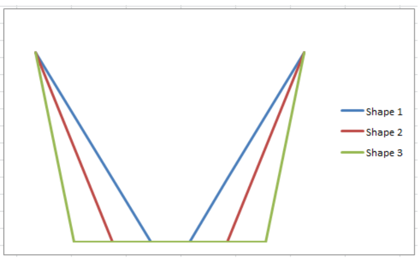

Figure 5.3: Sketches of different assumed cross-sectional shapes

To check which profile is the best to use in further simulations, 59 locations have been selected, and the water levels from the HEC-RAS model are noted. Then for those locations, the water levels from the three different simulations using Delft3D are compared. A useful method to analyze that is by checking the sum of the squared differences between the Delft3D values and the HEC-RAS values. By doing so, it turns out that shape 2 represents the situation the best. Figure 5.4 shows how the differences intervals are distributed.

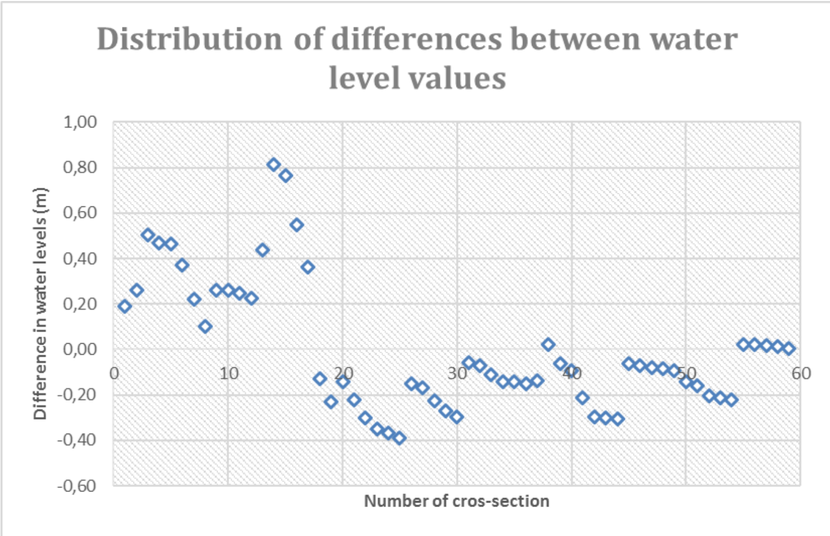

Figure 5.4: Distribution of differences between water level values

Apparently, 50% of the differences between the Delft3D model and the HEC-RAS model are below 20cm, and 80% are below 50cm. So it can be concluded that by using shape 2, a hydrodynamic model can be set up which is similar to the situation of the Volta Delta.

It should be noted that the Ghanaian paper did not mention whether the authors validated their model by comparing the water levels from their model with real water levels. Therefore, it can not be said that the Delft3D model represents the hydrodynamic behavior of the river quantitatively.

5.3. ADDING MORPHOLOGY AND VALIDATION OF THE MODEL

According to the previous section, there are differences between the water levels according to the Delft3D model and the HEC-RAS model. Although the differences are not too much, these differences mean that the assumed cross-sectional shape for the Volta river is not correct. This uncertainty needs to be taken into account when looking at the results. However, this model can still be used for other cases since the results between the two models are not extremely different.

In the next step, morphology is added to the model. With available information from the literature about the Volta River, a model will be constructed and calibrated in such a way that the model represents the Volta River. In subsection 5.3.1. the available information from the literature is described. This is mainly information about the yearly transported amount of sediment (types) to the ocean. An essential but missing part of the data is the distribution of sediment load during a year. Subsection 5.3.2. describes a method that aims to estimate this distribution in time.

5.3.1. TRANSPORTED SEDIMENT IN THE PRE-DAM SITUATION

In the literature, different researches can be found that tried to estimate the amount of transported sediment in the Volta River. The estimation of the yearly sediment load before building the Akosombo Dam made by Boateng et al. (2012) [16] is the starting point for the calibration of the Delft3D morphodynamic model.

Boateng et al. (2012) [16] estimated the amount of transported sediment to the ocean for different rivers in Ghana. The estimations were based on observed mathematical relationship between the sediment yield and the catchment area. For suspended sediment, the authors came up with this equation:

$$S_{suspend} = 20.722 - 1,127 \times a + 0,001422 \times a^2 \tag{5.1}$$

where $S_{suspend}$ is the suspended sediment yield in tonnes per year, and a is the catchment area in km^2.

The catchment area of the Volta River is 394100 km^2 which means that the transported amount of suspended sediment each year is about 220 million tonnes. By using a conversion scale of 1.602 tonnes to 1,0 m^3 the authors converted the estimated sediment load from tonnes per year to cubic meters per year. This turns out to be 137 million cubic meters.

The authors also estimated the mean bed load using the formula:

$$S_{bed} = -520,6 + 1.961 \times d \tag{5.2}$$

where S_{bed} is the bed load in cubic meter per year, and d is the mean annual water discharge calculated with the formula:

$$d = -422 + 3.050 \times a \tag{5.3}$$

with d in m^3/s and a the catchment area in km^2.

With these formulas, it turns out that the transported amount of sediment as bed load is about 2,3 million m^3 per year.

Apparently, most of the sediment was transported as suspended sediment load. In their paper, the authors mention that not all the suspended sediment load contributes to the beach condition. This is because fine particles (grain size <63 μm) are carried away offshore or to estuaries and marshes down-drift [39] [40]. According to Ayibotele and Tuffour-Darko (1979), [41] 50% of suspended sediment load in Ghanaian rivers have grain sizes lower than 63μm.

The table below shows different rivers and their estimated values for suspended and bed sediment load. The values in columns 3 and 4 are calculated using formulas 5.1 and 5.2 respectively. The values in column 5 are obtained by adding up the values in column 3 and 4. The values in the last column are obtained by adding half of the values of column 3 by the values in column 4.

The rows of the Volta River, the Pra River, and the Ankobra River contains double values. The values with the "@" sign are measured values (Collins and Evans (1986) [42] for the Volta River and Amisigo and Akrasi (1997) [43] for the Pra and Ankobra Rivers). It can be seen that for the Volta River the values for the mean suspended sediment are very different. This is because the measured values are from 1986 after building the dam). For the Pra River, the measured value for suspended sediment is very close to the estimated value. For the Ankobra River, it is less accurate but still in the same order of magnitude.

While for the three above mentioned rivers the suspended sediment load was measured, the bed load was only estimated by assuming it is 10% of the suspended sediment load. Since this assumption is not accurate according to different authors based on empirical evidence ([44], [45], [46], [47], [26], [40]), it can be assumed that the estimated values using equation 5.2 is more accurate than values of 10% of the suspended sediment load. This was also the reason for the authors to come up with equation 5.2 [16].

Names of rivers, streams and lagoons	Catchment area (km^2)	Mean Suspended sediment yield (m^3/a)	Mean bed load yield (m^3/a)	Total fluvial sediment yield (m^3/a)	Sediment yield significant to the beach (m^3/a)
Keta Logoon (Tordzie River)	2228	15.774	11.938	27.712	19.825
Volta River	394.100	@11.808.240 137.599.395	*1.180.824 2.355.744	@12.989.064 139.955.140	@7.084.940 71.155.442
Sakumo Lagoon (Densu River)	2550	16.913	13.864	30.777	22.321
Narkwa Lagoon (Okyi Nakwa River)	1500	14.727	7584	22.311	14.948
Amisa Lagoon (Okyi Amisa River)	1370	13.444	6807	20.251	13.529
Pra River	22.714	@485.660 454.912	*48.566 134.466	@534.226 589.379	@291.400 361.922
Ankobra River	8461	@106.588 70.528	*10.659 49.218	@117.247 119.746	@63.950 84.482
Dwuen Lagoon (Tano River)	26.489	617.128	157.045	774.173	465.609
Total, Pre-Volta Dam	464.984	138.855.619	2.753.443	141.609.048	72.181.265
Total, Post-Volta Dam	464.984	13.064.464	1.578.523	14.642.972	8.110.763

Table 5.1: A list containing some coastal rivers in Ghana with their sediment yield, from Boateng et al. (2012) [16]

In the construction of the model, a distinction is made between cohesive and non-cohesive materials. The most important reason for this is that mainly non-cohesive materials contribute to the beach. Another reason is that both types of sediment materials behave differently. Since cohesive material does not contribute to the beach (which is the main focus of this thesis), there will be no distinction made between different types of cohesive materials, e.g., silt and clay. This is also not possible because there is no information available about the different types of cohesive materials.

5.3.2. DISTRIBUTION OF SEDIMENT LOAD IN TIME

While the total amount of the yearly sediment load in the pre-dam situation is known, the distribution of the sediment load (and sediment concentrations) in time is not known. Assuming a constant load and a constant sediment concentration during the year is not realistic. It is expected that sediment concentrations are higher during periods of high discharge. Since this is an essential part of the model for validation with no available data about it, this distribution should be estimated. In figure 5.5 the discharge pattern during a year is given in blue. Also, the estimated concentration patterns of sand (non-cohesive material) and mud (cohesive material) are plotted in yellow and red respectively. Below, the derivation that led to this pattern is described in details.

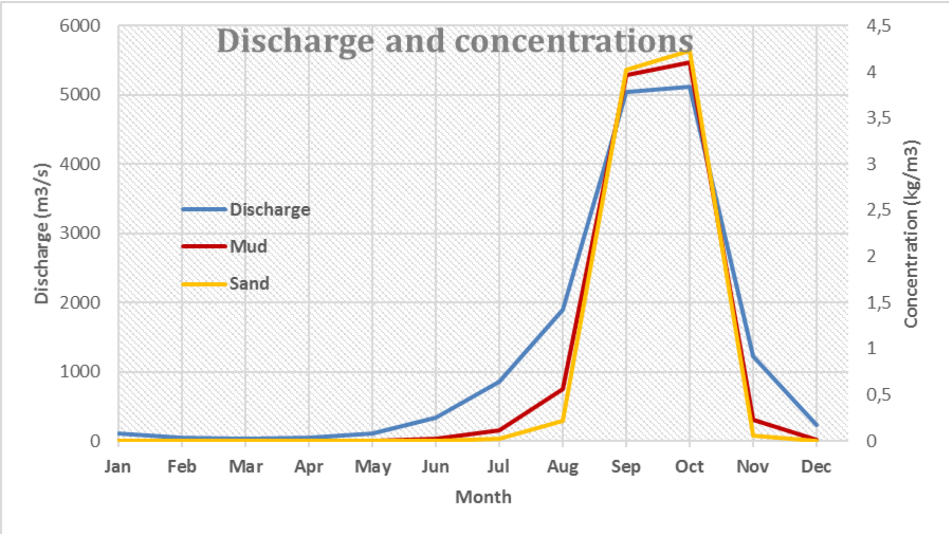

Figure 5.5: Discharge and concentration pattern for mud and sand during a year

RELATION BETWEEN CONCENTRATIONS AND DISCHARGE

For cohesive sediment, the sediment concentration is proportional to the discharge squared in the form:

$$C = \alpha Q^2 \tag{5.4}$$

For non-cohesive sediment, the concentration is proportional to the discharge to the power three:

$$C = \beta Q^3 \tag{5.5}$$

Proof for these equations is shown later in this section. According to Ofori et al. [35], the transported amount of sediment can be estimated using the formula:

$$Q_s = 0,0864 C_s Q_w \tag{5.6}$$

where Q_s is the amount of transported sediment in tonnes/day, C_s is the sediment concentration in mg/l and Q_w the discharge in m^3/s. By looking for values for α and β in order to estimate the concentrations for each corresponding discharge and demanding that the total amount of sediment in a year is 220 million tonnes (Boateng et. al [16]) one can find a value of $1,56 \cdot 10^{-7}$ for α and $3,14 \cdot 10^{-11}$ for β so that:

$$C_{mud} = 1,56 \cdot 10^{-7} Q^2 \tag{5.7}$$

and

$$C_{sand} = 3,14 \cdot 10^{-11} Q^3 \tag{5.8}$$

with C in kg/m^3 and Q in m^3/s.

With these relations, the concentrations for mud and sand during different discharges around the year can be calculated for construction of a model that can be used for validation.

PROOF OF THE RELATIONSHIP BETWEEN MUD CONCENTRATIONS AND DISCHARGE

Equation 5.4 in subsection 5.3.2 states that the concentration is proportional to the discharge squared in the form $C_{mud} = \alpha Q^2$. Below, this formula is derived, and a value of α is calculated to compare it with the α value of $1,56 \cdot 10^{-7}$ that is mentioned in the previous section.

The change in concentration in a certain area in time is the difference between eroded material from that area and deposited material in it. In formula form this can be written like this:

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = E - D \tag{5.9}$$

where u is the flow velocity and E and D the erosion and deposition fluxes respectively.
Both in in $kgm^{-2}s^{-1}$. Using the Partheniades-Krone formulations [31], the erosion flux can be written as:

$$E = M\left(\frac{\tau}{\tau_{cr,e}} - 1\right) \tag{5.10}$$

where M is the erosion parameter (in $kgm^{-2}s^{-1}$), τ the bed shear stress and $\tau_{cr,e}$ the critical erosion bed shear stress (in N/m^2). The deposition flux can be written as:

$$D = w_s C\left(1 - \frac{\tau}{\tau_{cr,d}}\right) \tag{5.11}$$

where w_s is the (hindered) fall velocity (m/s), C the mean sediment concentration in the water column (kg/m^3) and $\tau_{cr,d}$ the critical deposition bed shear stress (N/m^2).

In order to find an expression for the concentration, the specific case of a constant concentration is considered. In that case the erosion flux is equal to the deposition flux ($E = D$) and so:

$$M\left(\frac{\tau}{\tau_{cr,e}} - 1\right) = w_s C\left(1 - \frac{\tau}{\tau_{cr,d}}\right) \tag{5.12}$$

This can be regrouped as follows:

$$\frac{M}{\tau_{cr,e}}\tau - M = C\left(w_s - \frac{w_s}{\tau_{cr,d}}\tau\right) \tag{5.13}$$

and expressed in this form:

$$C = \frac{\frac{M}{\tau_{cr,e}}\tau - M}{w_s - \frac{w_s}{\tau_{cr,d}}\tau} \tag{5.14}$$

which can be simplified to:

$$C = \frac{\gamma\tau - M}{w_s - \delta\tau} \tag{5.15}$$

with $\gamma = \frac{M}{\tau_{cr,e}}$ and $\delta = \frac{w_s}{\tau_{cr,d}}$. The bed shear stress τ due to river flow can be written as:

$$\frac{g\rho_o \vec{U}|\vec{U}|}{C_{2D}^2} \tag{5.16}$$

with g is the gravitational acceleration (m/s^2), ρ_o the water density (kg/m^3), C_{2D} the 2D-Chézy coefficient ($\sqrt{m}/s$)and U the depth-averaged horizontal velocity (m/s). By using the equation $Q = UA$ (discharge is flow velocity multiplied by cross-sectional area), the equation for the bed shear stress can be written as:

$$\tau = \epsilon Q^2 \tag{5.17}$$

where

$$\epsilon = \frac{g\rho_o}{C_{2D}^2 A^2} \tag{5.18}$$

The Chézy coefficient can be calculated by $\frac{\sqrt[6]{H}}{n} = \frac{\sqrt[6]{7,40}}{0,033} = 42$, where H is the average water depth for the river and n is the manning coefficient (0,033 according to Logan et. al [15]). A is the average cross-sectional area of the river which is $2.775m^2$.

Substituting the minimum and maximum discharge values of $36m^3/s$ and $5.128m^3/s$ respectively in equation 5.17 shows the range of the bed shear stress in the river:

$$\frac{10\times 1000}{42^2\times 2775^2}\times 36^2 < \tau < \frac{10\times 1000}{42^2\times 2775^2}\times 5128^2 = \tag{5.19}$$

$$0,00095 < \tau < 19,33N/m^2 \tag{5.20}$$

The M term and the $\delta\tau$ term in equation 5.15 can be neglected. This can be shown by filling in common values for the terms in the formula. For example $M = 0,0001$, $\tau_{cr,e} = 0,5$, $w_s = 0,00025$ and $\tau_{cr,d} = 1.000$ (these are also the default setting values for these parameters in Delft3D). By doing so, and by considering the range of the bed shear stress (equation 5.20), it can be seen that the $\delta\tau$ term in the denominator can be neglected because the order of magnitude of the fall velocity term w_s is much larger. For the numerator, for $\tau > \tau_{cr,e}$, the order of magnitude of the $\gamma\tau$ term becomes larger then that of the M term. Therefore this term can be neglected as well. So by simplifying formula 5.14, it can be shown that the concentration is proportional to the shear stress:

$$C = \frac{\frac{M}{\tau_{cr,e}}}{w_s}\tau = \tag{5.21}$$

$$C = \frac{\frac{0,0001}{0,5}}{0,00025}\tau \tag{5.22}$$

with M, $\tau_{cr,e}$ and w_s as constant values. When substituting 5.17 for τ, C be expressed as: $C = \alpha Q^2$:

$$C = \frac{\frac{M}{\tau_{cr,e}}}{w_s}\frac{g\rho_o}{C_{2D}^2 A^2}Q^2 = \tag{5.23}$$

Filling in the parameters in the previous equation leads to the formula:

$$C = \frac{\frac{0,0001}{0,5}}{0,00025}\times\frac{10\times 1.000}{42^2\times 2.775^2}\times Q^2 = \tag{5.24}$$

$$C = 5,88\cdot 10^{-7}Q^2 \tag{5.25}$$

The derived value for α is in the same order of magnitude as the value of $1,56\cdot 10^{-7}$ that is found by using relations like equation 5.6. The value estimated with the unknown parameters for M, w_s and $\tau_{cr,e}$ however, is almost four times as high as the value estimated using the relationship between sediment load and concentration. This means that the parameters M, w_s and $\tau_{cr,e}$ needs to be adjusted in such a way that equation 5.23 gives a value of $1,56\cdot 10^{-7}$ for α. This can be done by decreasing the erosion parameter or increasing the values for fall velocity and critical erosion bed shear stress. This means that the riverbed seems to be less erosive compared with the default settings situation.

RELATION BETWEEN SAND CONCENTRATIONS AND DISCHARGE

For non-cohesive sediments like sand, the deposition flux is defined as:

$$D = \rho_s w_s C \tag{5.26}$$

where ρ_s is the sediment density, w_s the (hindered) fall velocity and C the sediment concentration.

The erosion or pick-up flux is calculated with empirical expressions and is often presented in a dimensionless shape using Φ_p:

$$\Phi_p = \frac{E}{\rho_s \sqrt{g\Delta d}} \tag{5.27}$$

A well known expression for calculating Φ_p is the equation of Van Rijn (1984) [48]:

$$\Phi_p = 0,00033 D_*^{0,3} \left(\frac{\theta - \theta_{cr}}{\theta_{cr}}\right)^{1.5} \tag{5.28}$$

with D_* defined as:

$$D_* = d\sqrt[3]{\frac{\Delta g}{v^2}} \tag{5.29}$$

d the representative grain size diameter, g the gravitational acceleration, v the water viscosity and $\Delta = \frac{\rho_s - \rho_w}{\rho_w}$, with ρ_s and ρ_w the specific density of sediment and water respectively.

θ the dimensionless bed shear stress:

$$\theta = \frac{\tau}{(\rho_s - \rho_w) g d} \tag{5.30}$$

τ is the bed shear stress in N/m^2 and θ_{cr} the dimensionless critical Shields value.
A well-known equation for calculating this parameter is (Brownlie, 1981) [49]:

$$\theta_{cr} = 0,22 R_p^{-0,6} + 0,06 \exp(-17,77 R_p^{-0,6}) \tag{5.31}$$

R_p is the particle Reynolds number defined as $\dfrac{d\sqrt{\Delta g d}}{v}$. If there is no erosion or accretion then:

$$E = D \tag{5.32}$$

and so:

$$\Phi_p \rho_s \sqrt{g\Delta d} = \rho_s w_s C \tag{5.33}$$

Regrouping leads to:

$$C = \frac{\Phi_p \rho_s \sqrt{g\Delta d}}{\rho_s w_s} \tag{5.34}$$

Substituting Φ_p and take out ρ_s:

$$C = \frac{\sqrt{g\Delta d}}{w_s} 0,00033 D_*^{0,3} \left(\frac{\theta - \theta_{cr}}{\theta_{cr}}\right)^{1.5} \tag{5.35}$$

The term $\dfrac{\sqrt{g\Delta d}}{w_s}$ is constant and the D_* is constant as well. The term $\dfrac{\theta - \theta_{cr}}{\theta_{cr}}$ can be simplified by neglecting the θ_{cr} term in the nominator in order to find a relationship between concentration and discharge.

So equation 5.35 can be written as:

$$C = \frac{\sqrt{g\Delta d}}{w_s} 0,00033 D_*^{0,3} \left(\frac{1}{\theta_{cr}}\right)^{1.5} \theta^{1.5} \tag{5.36}$$

In equation 5.36 all the terms are constant except for θ. According to equation 5.30, θ depends on τ, the bed shear stress. Since the concentration needs to be expressed in terms of discharge, equation 5.17 is substituted in equation 5.36:

$$C = \frac{\sqrt{g\Delta d}}{w_s} 0,00033 D_*^{0,3} \left(\frac{1}{\theta_{cr}}\right)^{1,5} \left(\frac{1}{C^2 A^2 \Delta d}\right)^{1,5} Q^3 \tag{5.37}$$

The fall velocity w_s can be calculated using the method of Van Rijn (1993):

$$w_s = \frac{10v}{d}\left(\sqrt{1 + \frac{0,01\left(\frac{\rho_s}{\rho_w} - 1\right) gd}{v^2}} - 1\right) \tag{5.38}$$

with $v = 10^{-6}$, $d = 0,0006m$, $\rho_s = 2.650 kg/m^3$ and $\rho_w = 1.000 kg/m^3$ the fall velocity turns to be $0,048 m/s$ By filling in the value for the fall velocity in equation 5.37 and filling in other values, the equation becomes:

$$C = \frac{\sqrt{10 \times 1,65 \times 0,006}}{0,048} \times 0,00033 \times 15,27^{0,3} \times \left(\frac{1}{0,0319}\right)^{1,5} \times \left(\frac{1}{42^2 \times 2.775^2 \times 1,65 \times 0,0006}\right)^{1,5} \times Q^3 = \tag{5.39}$$

$$C = 2,1 \times 0,00075 \times 175 \times 2,0 \cdot 10^{-11} Q^3 = \tag{5.40}$$

$$C = 0,56 \cdot 10^{-11} Q^3 \tag{5.41}$$

By comparing equation 5.41 with equation 5.8 it can be said that the both values for β are in same order of magnitude. However, the calculated β value is five times smaller as the value mentioned in subsection 5.3.2. This means that some values in the used formulas are not estimated correctly and needs to be adjusted as well.

5.4. OTHER RELEVANT MODEL ASPECTS

In this section, other relevant aspects for the Delft3D model are discussed. In the constructed model, the upstream boundary condition is always a discharge from the dam. The downstream boundary condition is the water level in the ocean (figure 5.6). Initially, the time step that was used in the simulation was 0,5 minutes. Despite the fact that the simulations were stable, the results were good enough. When smaller time steps were used, different results came from the model. The time step was decreased until the results of different time steps were almost the same. This was at a time step of 0,125 minutes. For reliable results, a long simulation time was needed. In the simulations that were made, a simulation time of 10 years is used. To decrease the calculation time, a morphological scale factor of 12 was used.

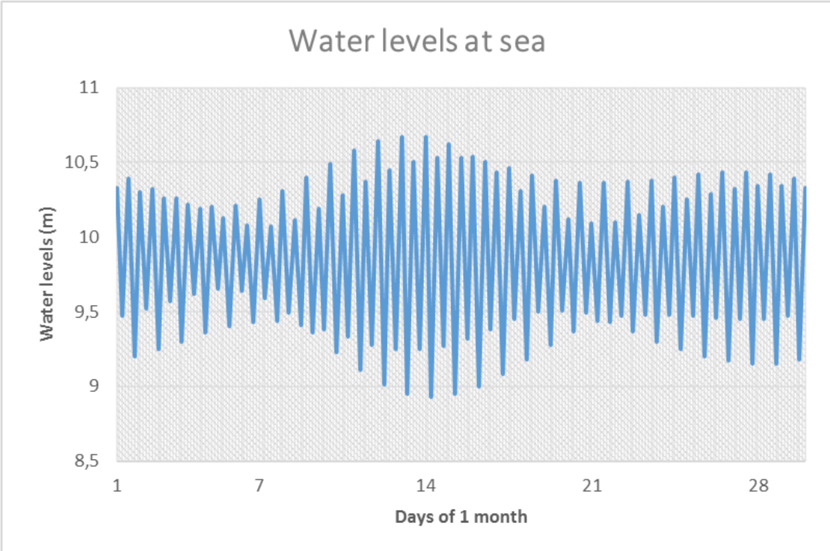

Figure 5.6: Water levels at sea during 1 month

In Delft3D, different transport formulas can be used for non-cohesive sediments like Van Rijn, Engelund-Hansen and Meyer-Peter-Muller. In the table below some of the available formulas are listed. The table also shows whether the formula makes a distinction between bedload and suspended load or not and whether waves are taken into account. Since for the Ghana case, the effect of waves is not taken into account, and there is a distinction made between bedload and suspended load, the formula of Van Rijn 1984 seems to be a suitable formula to use.

Formula	Sediment load	Waves
Ashida-Michiue (1974)	Total transport	No
Bijker (1971)	Bedload + suspended	Yes
Engelund-Hansen (1967)	Total transport	No
Gaeuman et al. (2009)	Bedload	No
Meyer-Peter-Muller (1948)	Total transport	No
Soulsby	Bedload + suspended	Yes
Soulsby/ Van Rijn	Bedload + suspended	Yes
Van Rijn (1984)	Bedload + suspended	No
Van Rijn (1993)	Bedload + suspended	Yes
Wilcock-Crowe (2003)	Bedload	No

Finally, aspects like bed slope effects and secondary flow (spiral motions) in river bends are accounted for by using the corresponding ’keywords’ for them in Delft3D.

5.5. VALIDATION OF THE MODEL

Despite some uncertain parameters due to the lack of data, the model can still be validated to check the reliability of it. This can be done by checking the amount of mean annual transported sediment to the ocean. According to Boateng et al. (2012) [16], that is in total 220 million tonnes per year. Assuming that half of it is cohesive material and the other half non-cohesive material [41], this means that for each type of sediment the transported amount of sediment is approximately 110 million tonnes per year.

First, a model is constructed with the pre-dam conditions with a simulation time of 10 years. Then the transported sediment to the ocean is checked and compared with the values from the literature. According to the Delft3D model, the transported amount of non-cohesive material is 97 million tonnes per year. This value is very accurate compared with the value according to Boateng et al. (2012). However, the sediment load of cohesive material according to the model turns out to be 47 million tonnes per year. This is about half of the value according to Boateng et al. (2012).

The simulation time for the model used for validation is ten years. This was done in order to make sure that the system was adapted to the situation and that it was in equilibrium. In figure 5.7 a long profile of the river bed can be seen for different years. It can be seen that in the first years the river bed levels are increasing and that the bed slope gets steeper. After some years the bed levels do not show much variation.

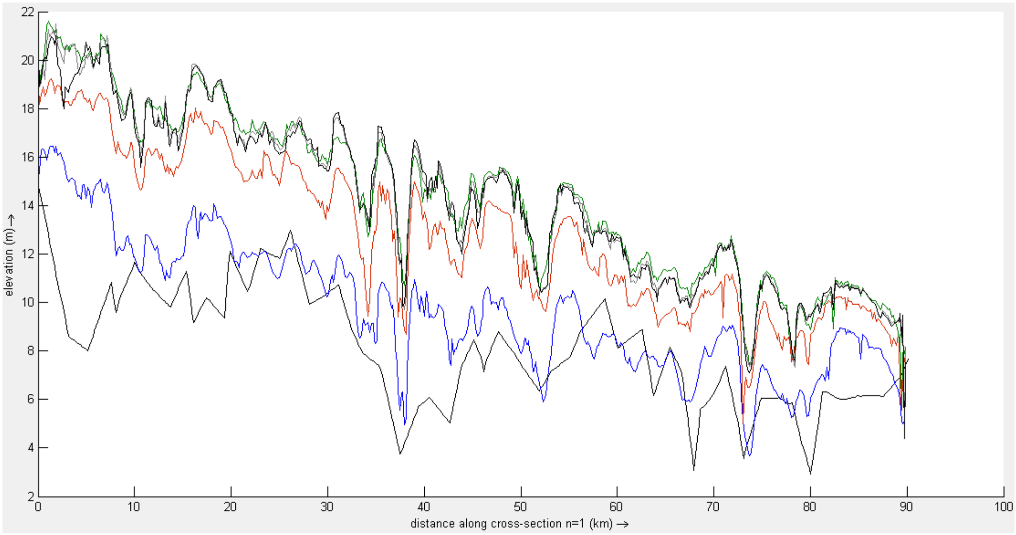

Figure 5.7: Bed levels after 1 year (blue), 3 years (red), 5 years (green), 7 years (grey) and 9 years (black)

With this result, the simulations could start to investigate the effects of the modified dam operations on the river's behavior. The next section describes that in detail.

5.6. SIMULATION SCHEMES

This section describes the way the simulations were done. There are three sets of simulation, and each of them considers one aspect of the modified dam operation scenario. Subsection 5.6.1. describes the simulation set for peak discharges, subsection 5.6.2. describes the simulation set for extra incoming sediments from the reservoir and finally subsection 5.6.3. deals with the simulation set for peak duration.

5.6.1. PEAK DISCHARGES

For this simulation set, the effect of different peak discharges is investigated. These simulations aim to check what the effects will be if the discharge regime of the Akosombo Dam will change. An important side note here is that only the discharge will change. This means that no extra sediment load will be added to the river from upstream. This sounds realistic since it was shown that modifying the dam operations to let the dam be operated as a run-of-the-river dam, will not transport sediment from the reservoir. Also, the Akosombo Dam has no low gates for sediments to be transported through them. This means that extra sediment that will be transported to the ocean will be from the river bed. For this simulation set, eight simulations were made for discharges of 1500, 2000, 2500, 3000, 35000, 4000, 4500 and 5000 m^3/s.

Since the aim of these simulations is to investigate only the effects of the peak discharges, the discharge during the year must be the same for all simulations: there should be only one varying variable 5.8. The results, however, from these simulations would not be any surprise. It is expected that a higher discharge will lead to more sediment load and more bed erosion. However, these results are still important to know what the order of magnitude is of the impact on the river.

An interesting thing to do is to also change the discharge during the months where there is no discharge. In chapter three, four scenarios where mentioned that the Ghanaian researches have investigated. Because of that, the effect of these scenarios on sediment load and bed level changes are also investigated. So in total, there are two sets of simulations made: the first one is with changed peak discharges only with constant discharge outside of the peak month to investigate the effect of that en the second one with the four scenarios that were mentioned earlier to check the effects of these (figure 5.9)

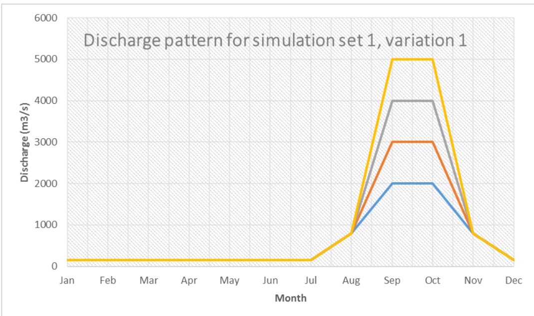

Figure 5.8: Discharge patterns with different peak discharges

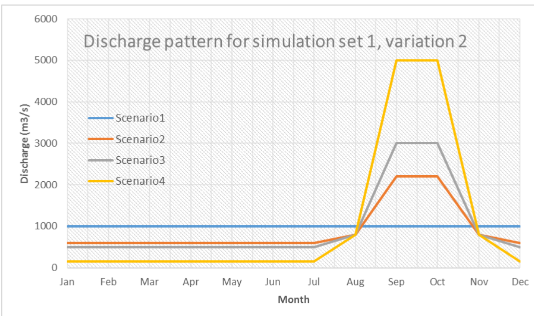

Figure 5.9: Discharge pattern of the designed four scenarios

5.6.2. INCOMING SEDIMENT LOAD

The second set of simulations considers the case where it is possible to transport sediment from the reservoir to the river. In the Ghanaian case, it would be not possible to transport sediment from the reservoir to the river by means of the described reservoir management methods in chapter three. Another possible way to transport the sediments would be dredging operations. For the Volta Delta, this might be expensive because of the large distance.

The second set of simulations does not consider how the sediment load is brought to the river and whether this is realistic (for the case study) or not. It is assumed that extra sediment load can be transported from the reservoir to the river. The focus of the simulations is not what happens upstream of the river but what happens in the river and how the river response to that extra sediment load.

In these simulation sets, the discharge is kept constant as it is now (1000 m^3/s) and the incoming sediment concentration is also constant during the year. Seven simulations are made with different sediment concentrations. These are 10, 20, 30, 40, 50, 60 and 70% of the averaged concentration value in the pre-dam situation.

5.6.3. PEAK DURATION

The last set of simulations considers the case where the duration of the peak discharge is changed. Five simulations are made to investigate this with a peak duration of 2, 3, 4, 5 and 6 weeks. In this set, the discharge during the other months is kept constant for all the simulations. Finally, two variations of sets are made: one with a peak discharge of 5000 $m^/s$ and the other with 3000 m^3/s.

Figure 5.10 shows a scheme with the simulation sets that are made. In the figure, two extra simulations can be seen which will be discussed later in the next chapter.

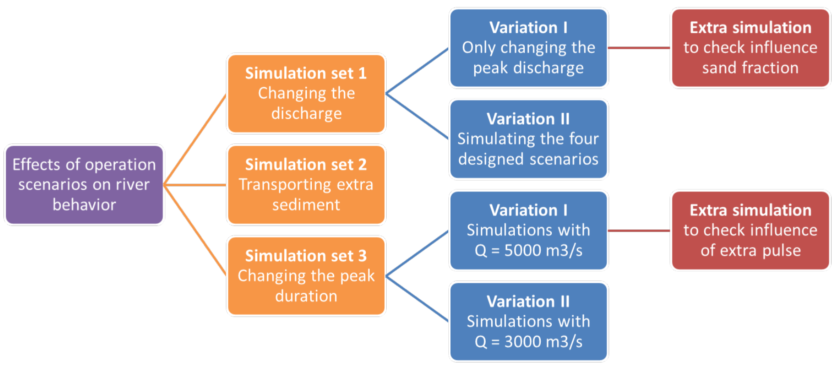

Figure 5.10: All the simulation sets summed up

6
Results

In this chapter, the results of the simulations for both the reservoir part and the river part are discussed. Section 6.1 describes the results from the reservoir simulation. As mentioned before in the introduction, the modified dam operation method is mainly meant for the downstream situation, and more effect downstream is expected compared to the reservoir part. Therefore, the results for the reservoir part will focus on the question of whether this method works for the upstream part or not and how can it be improved. Sections 6.2, 6.3 and 6.4 describe results for the river part for the three different simulations sets. When the results are shown, short remarks are made about the observed behavior. Two times, some of these short remarks is tested with an extra simulation to prove whether it is correct or not.

For the downstream part, the sediment load is different at different areas at different times during the year. To compare the results, the yearly averaged, width averaged value near the coast is checked. Figure 6.1 shows the model grid for the Volta River with the selected spot where the sediment load is checked. This spot is chosen because it is the nearest point towards the coast before the river channel gets wide (where the sediment transport is expected to drop because of that). In the selected cross-section, the sediment load also varies along the width because of variation in depth (see figure 6.2). For the results, the width averaged value is calculated and compared with other simulations. For the bed level changes, the width averaged values are also calculated along the river (figure 6.3). However, since it is hard to compare different graphs like that, a linear trend line for each graph is drawn so that it would be easier to compare the results.

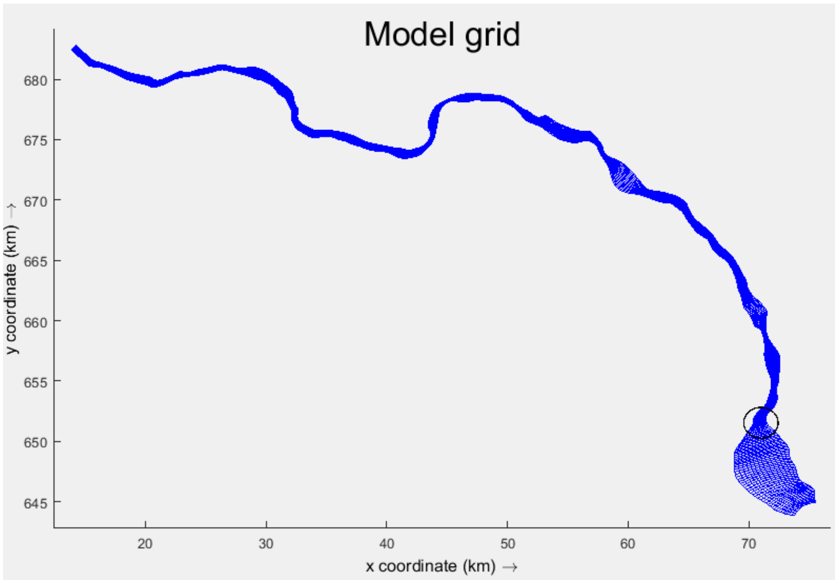

Figure 6.1: The model grid with the selected place for checking the sediment load

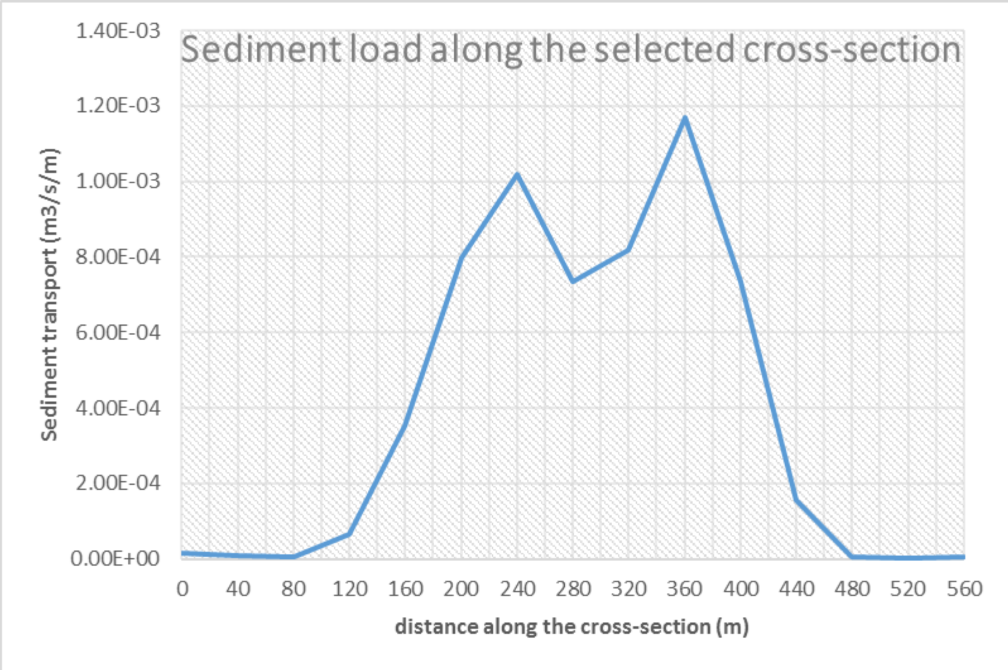

Figure 6.2: The sediment load along the selected cross-section

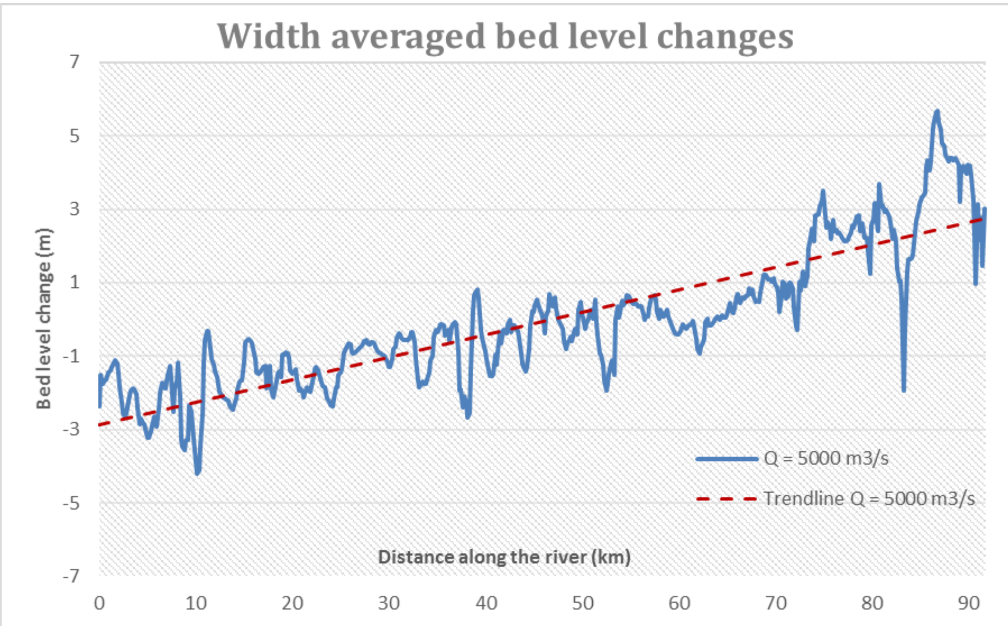

Figure 6.3: The width averaged bed level changes and the corresponding trendline

6.1. THE RESERVOIR PART

Figure 6.4 shows the total sediment load along the lake for the first (upstream) 10 km for the first two simulations: simulation 1 has a constant discharge through the dam as in the current situation and simulation 2 has the same discharge as what is coming in the reservoir. It can be seen that the only difference between the two simulations can be found in the first 2000 meters from upstream. After that, the sediment load for both simulations drops down to a negligible rate. The dam itself is located 400 km further downstream. Figure 6.5 shows the development of the transport rate further downstream on a logarithmic scale. It can be seen that once the transport rate has reached an order of magnitude of power minus 19, it continue to decrease slowly further downstream to a certain point where it increases again. This is because of the effects of the dam discharge on that part of the reservoir. Despite that increase, there is hardly any clear differences to see between the two situations.

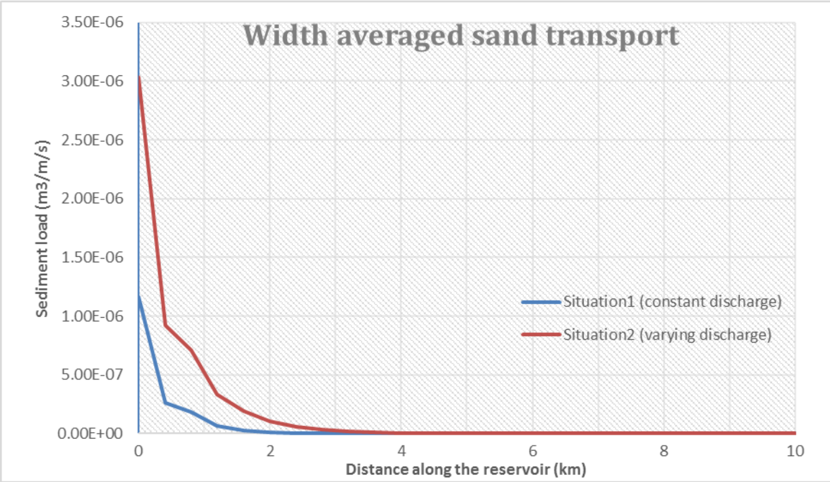

Figure 6.4: Sediment load in the reservoir for simulation 1 (constant discharge) and 2 (varying discharge) for the most upstream 10 km

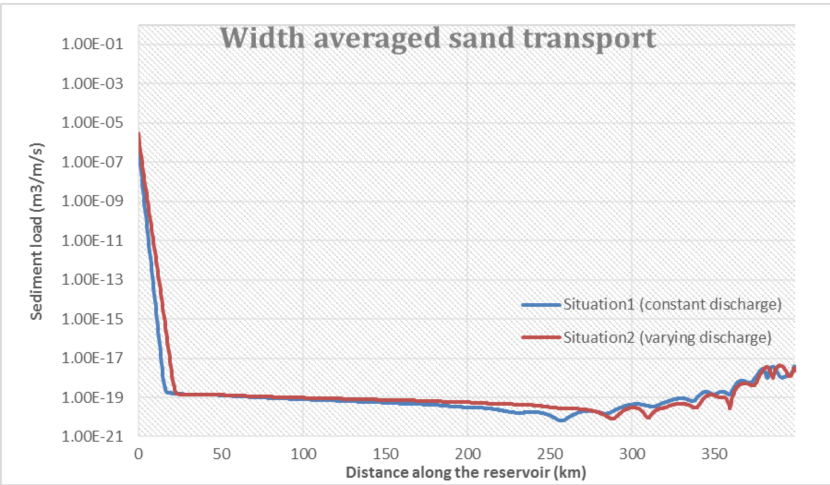

Figure 6.5: Sediment load in the reservoir for simulation 1 (constant discharge) and 2 (varying discharge)

Apparently, modifying the dam operations, do not affect the transport load. A reason for this is most likely the size of the reservoir. To check what characteristics may cause this, two extra simulation sets are made: in the first set, the width of the reservoir is decreased. In the new simulation, the reservoir is made narrower to a width which is the same as the previous width of the river. The results of the simulation for a case with a constant discharge and a discharge pattern which is the same as the incoming discharge are shown in figure 6.6. Compared with figure 6.4, there is more happening in the reservoir, especially in the most upstream and most downstream part.

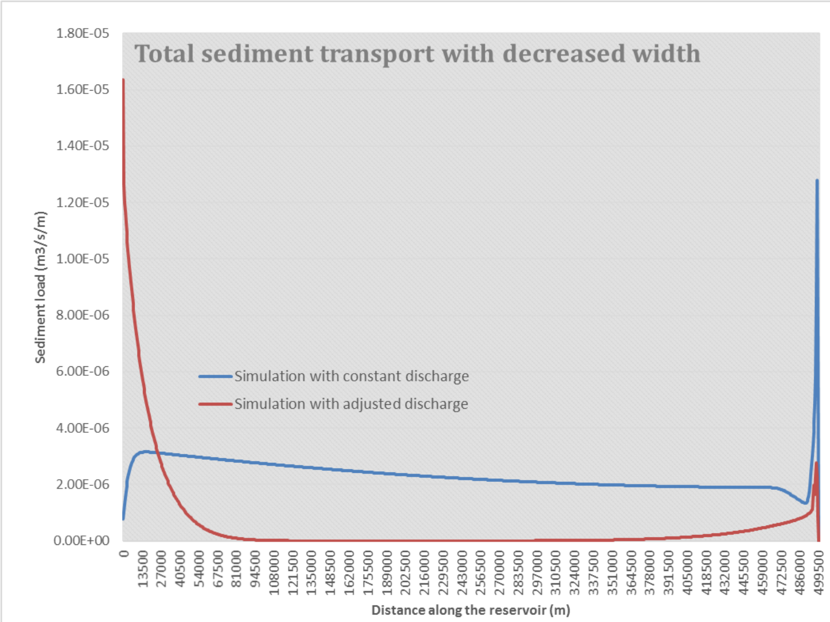

Figure 6.6: Sediment load in the reservoir with decreased width for simulation 3 (constant discharge) and 4 (varying discharge)

It can be seen that for the most parts of the reservoir, the current regime transports more sediments compared with the adjusted discharge scenario. Only for the most upstream part in the reservoir, the new discharge scenario causes more transport. For the most downstream part, it is the opposite: the current regime transports more sediment downstream. Since the interest of the operation scenario was to carry more sediment to the river, it can be said that this new solution will not provide that. On the contrary, it will decrease the transport to the river. This all is in the case of a narrow, deep reservoir.

For the last simulation set, the width of the reservoir is kept the same, but the depth is decreased. Figure 6.7 shows the sediment load for the last two simulations (with a constant discharge and a variable discharge) for the downstream part of the reservoir. It can be seen that for this case, the operation scenario causes more sediment transport.

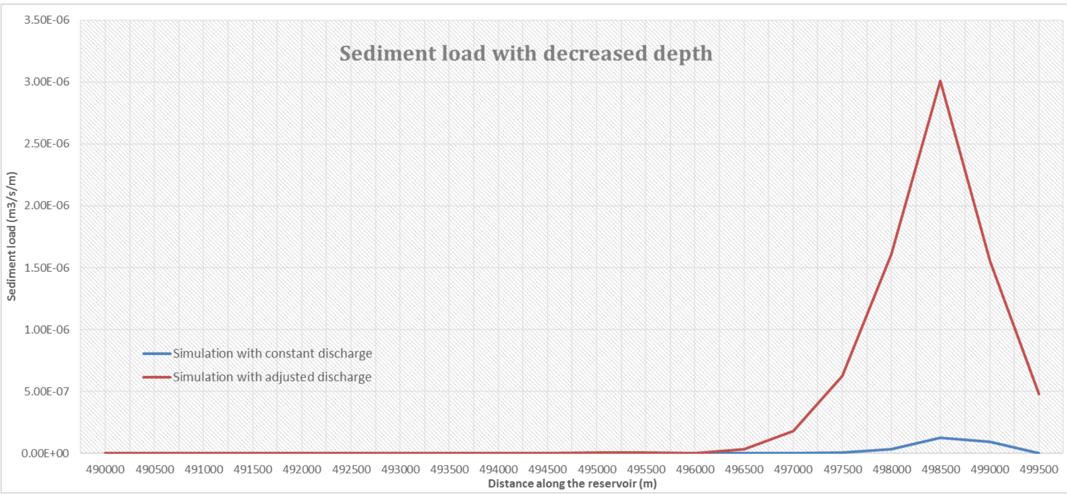

Figure 6.7: Sediment load in the reservoir with decreased depth for simulation 5 (constant discharge) and 6 (varying discharge)

So from these three simplified simulations, it can be said that operating the dam as a run-of-the-river dam for the Volta reservoir will not affect the sediment (transport) because of the reservoir characteristics. For a narrow reservoir, the operating regimes seem not to be beneficial for transporting more sediments. However, the method seems to suit a wide shallow reservoir.

6.2. THE RIVER PART: CHANGING THE DISCHARGES

6.2.1. VARIATION I: ONLY CHANGING THE PEAK DISCHARGE

Figure 6.8 shows the sediment load transported to the coast for different peak heights. During the other months, all the scenarios have the same discharge. In the graph both transport of sand and mud are shown. The graph shows two clear things: first of all, there is a linear relationship between the sediment load and the discharge and secondly more sand is transported than mud for each discharge. This might be because the bed contains more sand (63%) than mud (37%). While this might be one reason for the difference in load values, it is not the only reason. The ratio between sand and mud in the initial bed level is 63:37 = 1,7. The ratio between sand and mud transport is 4,4 for a discharge of 1500 m^3/s and increases to 4,7 for a discharge of 3000 m^3/s. To check this, a simulation is made with the same amount of sand and mud in the riverbed for a peak discharge of 5000 m^3/s.

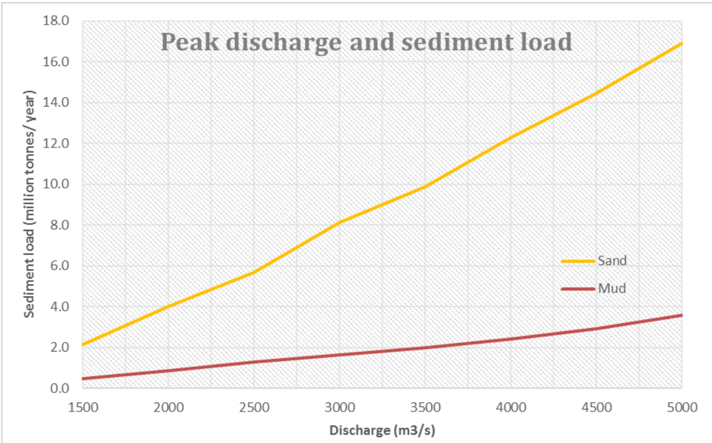

Figure 6.8: Relation between peak discharge and sediment load for sand and mud

Figure 6.9 shows the results of both simulations with 5000 m^3/s for the original sand/ mud ratio and the adjusted one. In the new situation, the total sediment load has dropped from 20,5 million tonnes to 17 million tonnes. That is a decrease of 17%. The amount of sand transported also fallen from 16,9 million tonnes to 12,9 million tonnes which is a decrease of 24%. In contrast to that, the mud transport has increased from 3,6 million tonnes to 4,1 million tonnes which means an increase of 14%. Finally, the ratio of sand to mud in the sediment load to the ocean has changed from 4,5 to 3,2. So it can be said that the river bed fractions play a role in the distribution of the sediment load to the coast, but there are more aspects than that which make it possible to transport more non-cohesive material than cohesive material by the same bed fraction. These aspects might be among others bed slope and material characteristics.

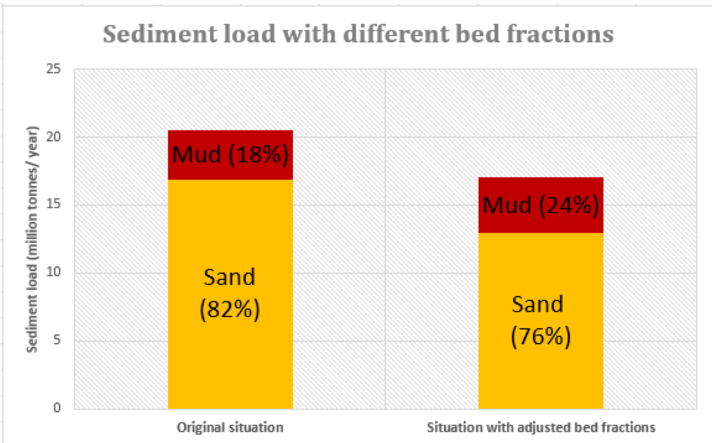

Figure 6.9: Sediment load with a discharge of 5000 m3/s for the original situation and the adjusted one

Figure 6.10 shows the trend lines of the bed level for different discharges. The bed level change from the simulation with the adjusted sand fraction is also included in the figure. There are some things that can be seen in this figure. First of all, it can be seen that for higher discharges during the peak, the bed is more eroded (upstream of the river). However, the erosion does not happen all along the river. From a certain point towards the ocean, the bed level is increasing. It is interesting to see that the trend lines are more or less parallel: this means that the bed slope of the river is not changing when the discharge is changed. Another interesting fact is that downstream, accretion is happening for the river bed. This might cause clogging of the Delta as it is happening now in the Volta Delta. Finally, it can be seen that the bed level changes for the simulation with adjusted sand fraction are in the same order of the simulation with the original fractions. The only clear difference between the two is the bed slope.

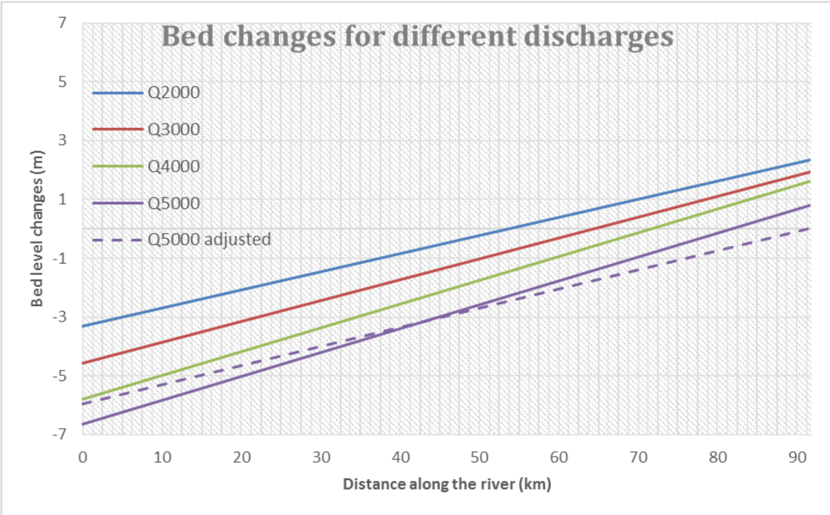

Figure 6.10: Trendlines of bed level changes for different peak discharges

6.2.2. VARIATION II: REALISTIC OPERATION SCENARIOS

Figure 6.11 shows the four scenarios that were described in chapter 3. The main difference between these scenarios and the ones in the previous subsection is that these scenarios have more or less the same volume of water that is discharged through the dam in one year. Figure 6.11 shows the sediment load for three operation scenarios. It can be seen that scenario 4 has caused the most sediment load. This is obviously because of the highest peak discharge compared with the other two. The results of scenario 2 and 3 are more interesting since the discharges are close to each other. Even though scenario 2 has a higher discharge during the year compared with scenario 3, it has a lower sediment load. Apparently, the peak discharge is more important for sediment transport compared with the constant discharge outside of the peak time.

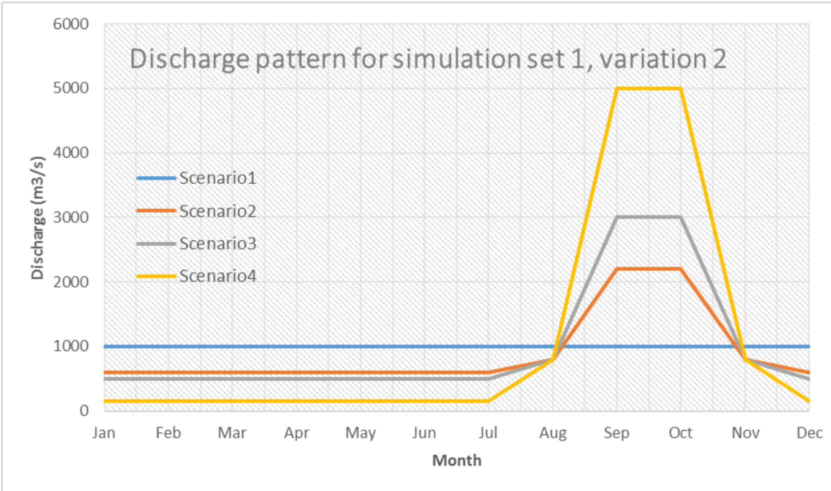

Figure 6.11: Discharge pattern of the designed four scenarios

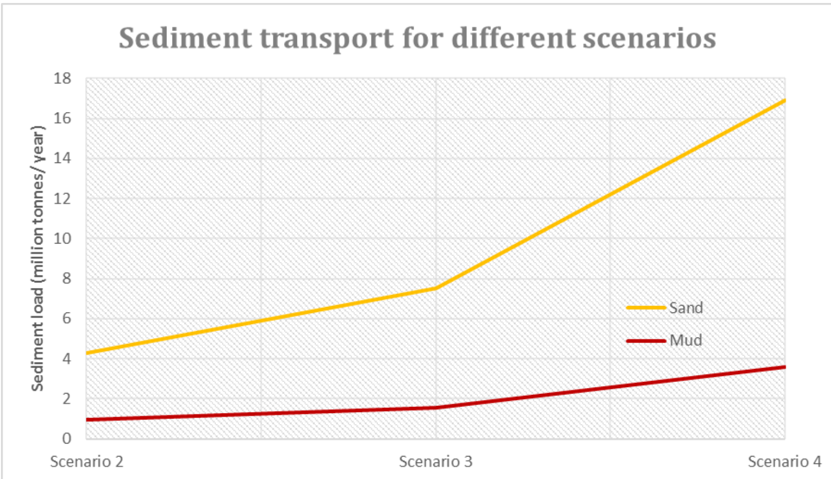

Figure 6.12: Sediment load for sand and mud for different scenarios

6.3. THE RIVER PART: EXTRA SEDIMENT LOAD FROM UPSTREAM

Figure 6.13 shows the sediment load transported to the coast for different concentrations (given as percentages of the pre-dam averaged concentration). It can be seen that there is an increasing trend line in the relation between the load and the concentration. While this is nothing special and the behavior was expected, it is more interestingly to look at the (relative) values for both concentration and load: while the concentration is increased until 70% of the previous averaged value (which means 70% of trapped sediment is dredged), only 15% of the pre-dam load is transported to the ocean if the discharge of the dam is kept constant the whole year. This scenario does not seem like a real one to apply. Dredging 70% of the sediment from upstream is likely to be an expensive operation and to do so for only extra 15% is not in proportion with the effort. If it is possible to dredge sediment from the reservoir to the river, then it would be interesting to investigate the option of introducing peaks again. Without that, this scenario would not be realistic.

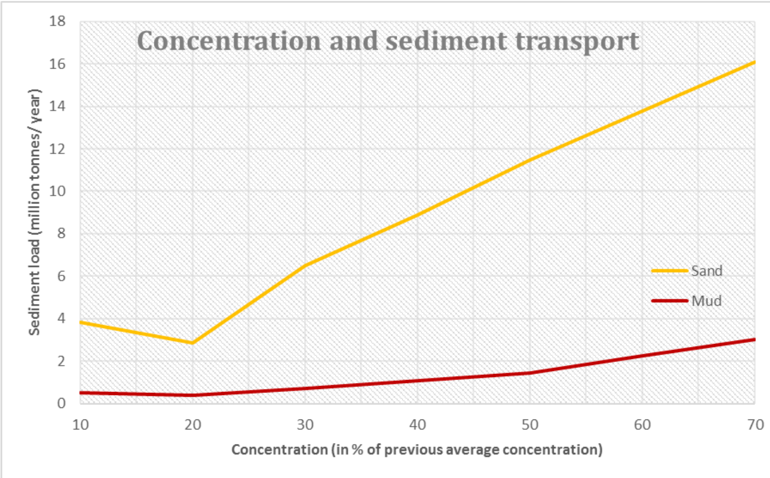

Figure 6.13: Relation between sediment concentration and sediment load for sand and mud during a constant discharge of 1000 m3/s

Figure 6.14 shows the trend lines of the bed level for different constant concentrations. It is very clear from the figure that the bed level is increasing. This result was expected since the transported amount of sediment to the ocean did not increase with the same rate as the increase in concentration. It means that most of the extra incoming sediment fall down to to the bottom. Apparently, a discharge of 1000 m^3/s is not strong enough to transport all the sediment to the ocean. When looking again to figure 6.14 it can be seen that by increasing the sediment concentration, more sediments are settled upstream. This means that for higher concentration under the same discharge of 1000 m^3/s, the bed slope is changing: it gets less steep.

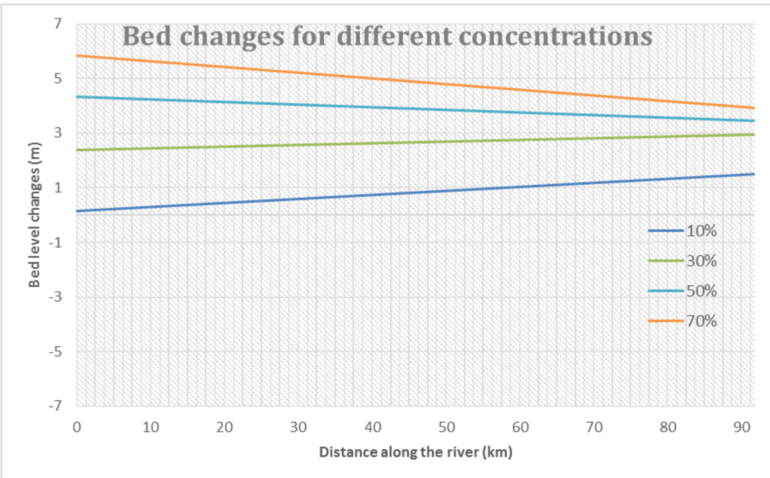

Figure 6.14: Trendlines of bed level changes for different sediment concentration

For the Ghana case, this result might be interesting because of the fact that 'only 10%' of the previously averaged concentration is needed to keep the riverbed more or less stable and prevent it from erosion. Dredging sediments from the reservoir to the river without creating pulses that transports those sediments to the ocean is not only disadvantageous because of the increased effort compared with the desired results, but also because it increases the river bed. This might be undesirable because then the water level is also going to increase and so more area along the river will be vulnerable to (artificial) floods.

6.4. THE RIVER PART: CHANGING THE PEAK DURATION

6.4.1. SEDIMENT LOAD AS A FUNCTION OF PEAK DURATION

Figure 6.15 shows the sediment load transported to the coast for a discharge of 5000 m^3/s for peak durations varying from 2 to 6 weeks. Even though there is more sediment transported during 6 weeks than 2 weeks, the relation between the load and duration is considered constant. This is because the increase in time (from 2 to 6 weeks) is much more than the increase in load (from 16 million ton to 17 million tonnes per year). This graph shows that (without incoming concentrations), the duration of the discharge pulse, or peak, does not matter for the sediment load to the ocean. To be sure about this, another simulation is made with different peak discharge (3000 m^3/s). In Figure 6.16 it can be seen that the sediment load is less, but that it is still more or less constant despite changing the peak duration.

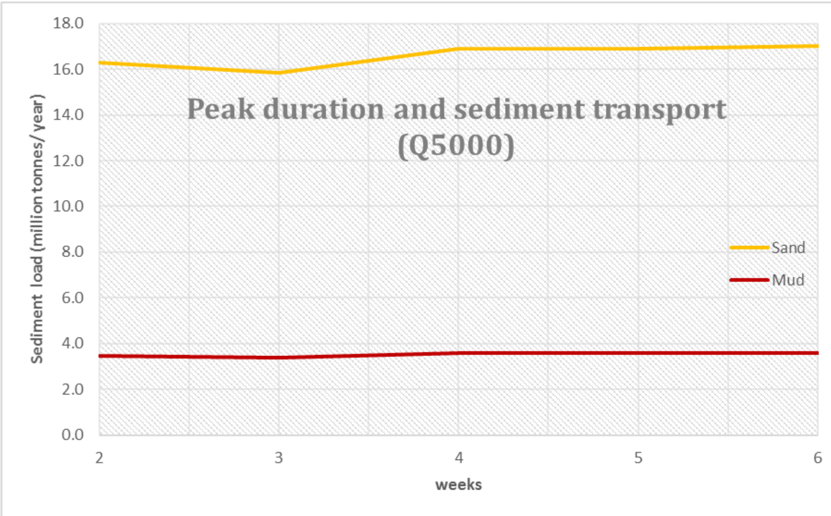

Figure 6.15: Relation between peak duration and sediment load for sand and mud during a peak discharge of 5000 m3/s

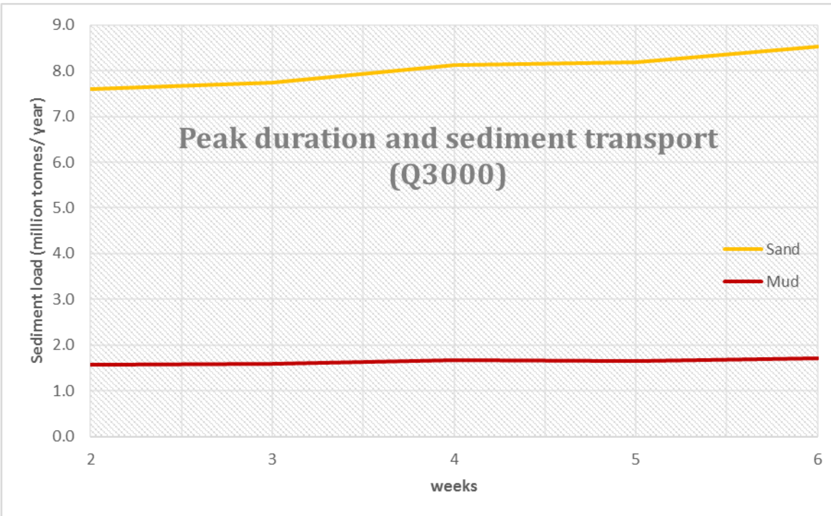

Figure 6.16: Relation between peak duration and sediment load for sand and mud during a peak discharge of 3000 m3/s

It is important to keep in mind that these simulations were made without incoming sediment load which means that despite that load capacity of the river was increased by increasing the peak duration, the amount of transport material did not change. So it is better not to say that the peak duration does not matter but rather to say that increasing the river capacity above a specific capacity that was sufficient for maximum sediment load does not change much about the situation. So in the Ghana case, apparently a shorter peak is sufficient to transport the maximum available material. Increasing the capacity without increasing the material will not change much.

One of the explanations for this might be that the erodible material is all eroded within one or two weeks. To check this, a plot is made of the cross-sectional area of the river. Figure 6.17 shows the average elevation levels for the initial river bed, the base level of the river bed and the river bed for the simulation with 2 and 6 weeks. It can be seen that not all the available bed material is eroded. So the assumption that all the bed material is eroded is not a valid one.

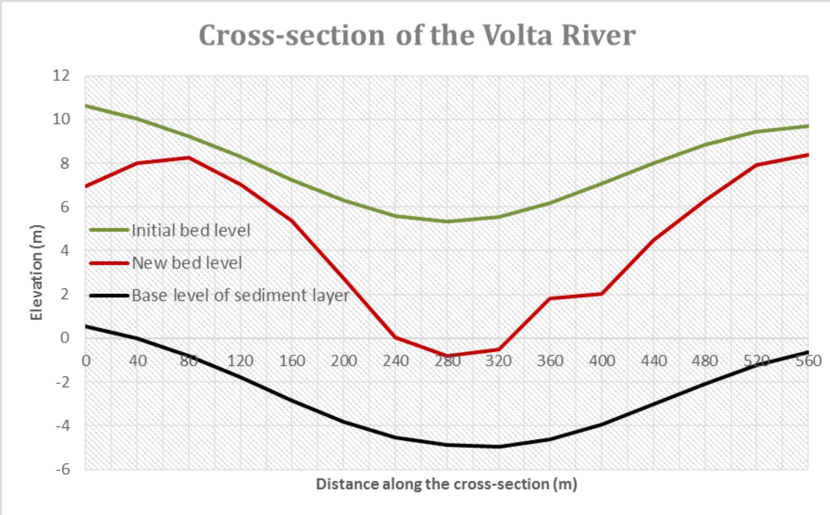

Figure 6.17: A cross-section including the new and old bed level and its base level

Another explanation for this behavior might be that there is something more important about the discharge pulses than their duration. It might be that the change in discharge in a particular time is more important than the length. To investigate this, a simulation is made with 2 discharge pulses of 2 weeks each (instead of 1 pulse of 4 weeks in the original situation). In terms of discharge, the situations are equal, but the last simulation has 2 times a period with increased discharge instead of only one. Figure 6.18 shows the results between a simulation with one pulse of 4 weeks in a year and 2 pules with 2 weeks in a year. It can be seen that doubling the number of peaks does not change much about the situation.

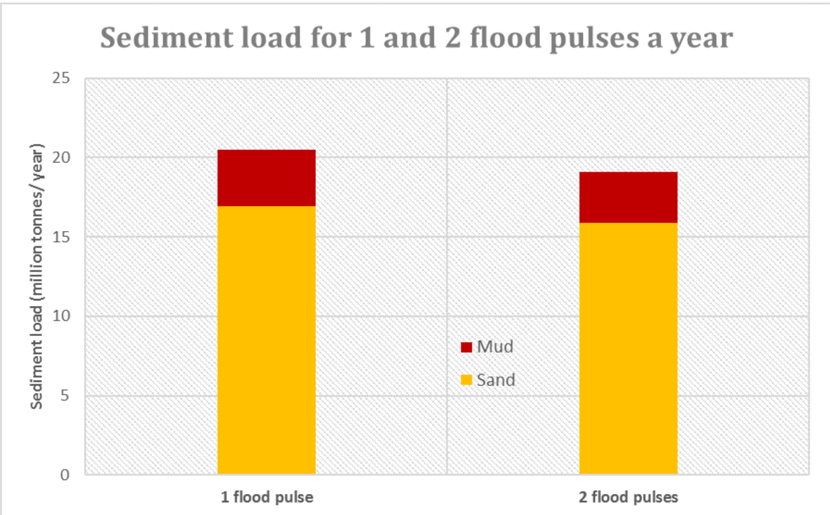

Figure 6.18: Sediment load for sand and mud for 1 and 2 pulses per year

Since the results for this scenario set might be important for decision makers since it makes the modified dam operations approach more realistic, it is interesting to figure out what might be the reason for this behavior. Appendix B describes a method that was carried out in order to find out why the river seems to be insensitive for the duration of a peak. As said before these results might be important for decision makers. The main disadvantage for this approach for Ghana was the decrease in energy production. However, if the time with a decreased energy production can be reduced, this method can become more interesting.

Figure 6.19 and figure 6.20 show the trend lines of the bed level for different durations of peaks. Since there are no much differences in sediment load between the simulations with different duration, there is also hardly any differences between the bed level changes as it can be seen in the figure. As said before this could be interesting for decision makers to investigate the case of a shorter peak duration and find out if it is economically attractive.

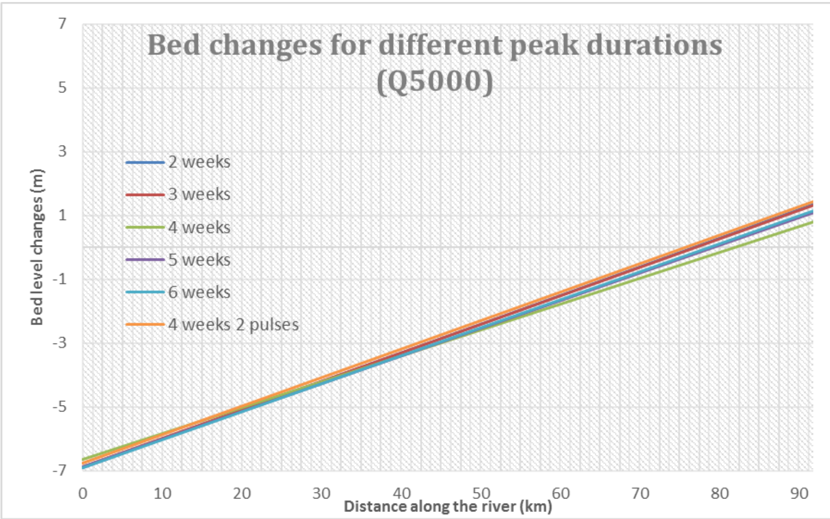

Figure 6.19: Trendlines of bed level changes for different peak durations during a peak discharge of 5000 m3/s

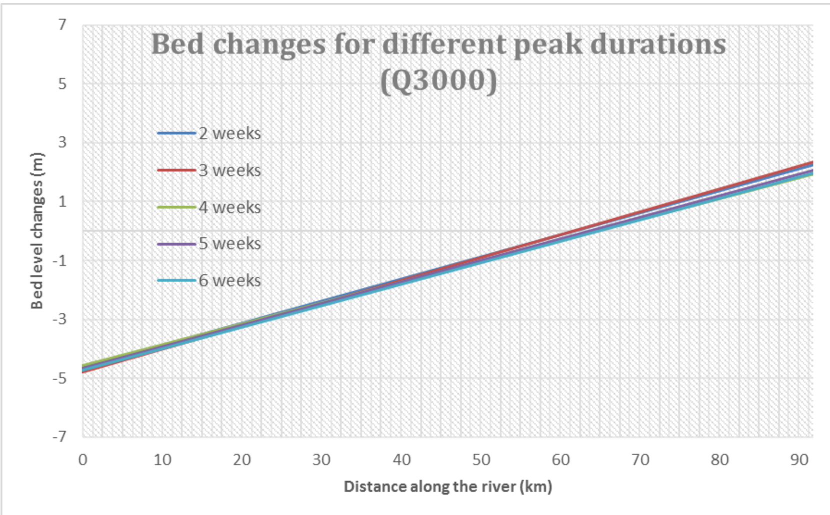

Figure 6.20: Trendlines of bed level changes for different peak durations during a peak discharge of 3000 m3/s

7

Discussion

Although dams can have many benefits for the area where they are located, they can also create negative effects on the surrounding area. Several techniques deal with problems caused by dams for the upstream area and the downstream area. For the upstream area, such methods are for example sluicing, flushing, and dredging operations. For the downstream area, fewer techniques are known. One technique that can help to improve the situation in the downstream part of the dam is modifying the dam operation. The aim of such method is to recreate artificial flood pulses in order to improve processes in the river. Unfortunately, there is not much known about the influence of such a technique for large reservoirs. This research aimed to make a start with the investigation of the effects of this technique on large reservoirs. Although this method is designed for the downstream area, this research also checked whether there is an effect on the reservoir part.

7.1. Summary and remarks about assumptions that are made

Due to the lack of data, some assumptions had to be done for both the reservoir part and the river part. Since there was no information available about the bathymetry of the Volta Lake, a simplified model was constructed that has the same averaged length, width, and depth. According to this model, the sediment transport in the reservoir is negligible because of the reservoir dimensions and changing the dam operation will hardly change anything about that. Despite the simplified shape of the reservoir in the model, it can be assumed that this result will also hold for the real situation because of the similarity in the large dimensions.

For the river part, more data was available, but still, there was a lack in some essential data such as the cross-sectional shape of the Volta River. For a hydrodynamic model, a trapezoidal shape was assumed. This assumption should be validated by comparing it with measured water levels values for the river. Since this data was also missing, the water levels according to the constructed model were compared with the water levels according to another model from the literature. 50% of the differences between the models were below 20 cm, and 80% was below 50 cm. Since it is not clear whether the authors of the model from the literature validated their results or not, it can not be exactly said how accurate the constructed hydrodynamic model is. More data about the Volta River is therefore needed to say something about the model.

For a morphodynamic model, another assumption was made about the sediment concentration in time in the river. With the known total amount of yearly sediment load, a relationship between concentration and discharge for cohesive and non-cohesive sediments was assumed. This was an essential part of the morphodynamic model that was needed to simulate sediment transport and bed level changes.

Before using the morphodynamic model, it was validated by making a simulation of the pre-dam river condition and comparing the sediment transport towards the coast according to the model with the known values from the literature. The model seemed to be very accurate for modeling non-cohesive sediments with a value for sediment load that is about 90% of the value according to the literature. For cohesive sediment the model seemed to be less accurate with a sediment load rate about half of the value that can be found in the literature. Here again, more data about the Volta River would help to construct more accurate models.

7.2. REMARKS ABOUT THE RESULTS

The most interesting result from the research is the fact that the peak duration has little impact on the river's behavior. This information might be important for decision makers because it can make this method more attractive. Appendix B gives a possible explanation for this behavior. However, this needs to be verified with more research. In the simulations that were done, only the duration with the peak discharge was changed. The time that was needed to increase the discharge to that peak discharge or to decrease it again was kept the same. It might be interesting as well to investigate whether it is possible to decrease this time as well and what the effects of that will be on the sediment transport. This will lower the time period where no extra power can be generated because of the turbines capacity limit and thus makes this method more interesting for decision makers.

Another important aspect is how extra added sediments from upstream behaves in the river. This research showed that extra sediment in the river without adjusting the discharge through the dam is not sufficient to transport the added material to the coast. Flood pulses are important as well. So in order for a sustainable solution, a combination should be created with the available sediment from the reservoir that can be transported, together with a short flood pulse. It might be interesting to investigate whether this combination will be the same for different reservoirs or whether it will depend on the specific situation.

8

Conclusion

In this research, the effects of modified dam operation scenarios were investigated for the areas upstream and downstream of a dam. For the river part downstream of the dam, three simulation sets were done using a Delft3D model to investigate three different aspects: the peak discharge of a recreated flood pulse, its duration and the sediment load entering the river from upstream.

8.1. The reservoir

Despite the lack of data during the construction of the models for investigation of the research questions, the obtained results can be used to draw some conclusions about the modified dam operations for large reservoirs. First of all, the method does not seem to be useful in routing sediments through the reservoir and dam for a large reservoir like the Volta Lake. For shallow and narrow reservoirs, this method seems to work. However, for these type of reservoirs other better-known techniques are available like sluicing operations.

8.2. The river: increasing the discharge

For the river part, this method seems to be worthy to investigate further. According to the results, there is a linear relationship between the peak discharge and the transported sediments towards the coast in case of no incoming sediments to the river.

The linear relation for sediments seems to apply for both cohesive and non-cohesive sediments. However, non-cohesive sediment appears to be more sensitive for discharge change and therefore more material is transported when the initial bed fractions are equal. A reason for this might be the material characteristics or the river bed slope.

If the peak discharge increases and there is no sediment flowing into the river, the river bed starts to degrade. The trend lines in the bed changes seem to have the same slope for different discharges which means that increasing the peak discharge through the dam does not affect the bed slope.

When the same water volume is distributed differently over the year, the peak discharge remains important for the sediment transport even when the release is higher during the other months.

8.3. The river: increasing incoming sediment load from upstream

Adding more sediment to the river from the reservoir might be an option. However, when the discharge in the river is kept constant, most of the incoming sediment will fall to the bottom and cause accretion in the river channel. Since this behavior is not desirable, it is crucial to include discharge pulses when bringing in more sediments from upstream.

8.4. THE RIVER: CHANGING THE PEAK DURATION

According to the results, there is hardly any change in sediment transport when the peak duration is changed. This is not because the available sediment layer in the model is eroded but probably because the river bed adapts to the discharge in such a way that the amount of transported sediment is almost the same for different peak durations. This result is interesting for decision makers since the same transport can be obtained with less change in the current regime.

8.5. THE RIVER: LOOKING FOR AN IDEAL SOLUTION

Based on these results, one can try to sketch an 'ideal' solution for the Volta Delta that might be applied elsewhere as well. In this ideal solution, the averaged water volume distributed over the year would be the same as it is now. Otherwise, the water levels in the reservoir would keep increasing or decreasing. The redistributed discharge pattern can have a shorter peak than it is proposed in the current scenarios. Because of that, the constant discharge during other months can be higher. This solution will increase the sediment load towards the coast, but it will cause bed degradation since this method will not provide extra sediment from the reservoir.

8.6. APPLYING MODIFIED DAM OPERATIONS FOR THE VOLTA DELTA

The focus of this thesis was on applying the modified dam operations method on large reservoirs. The reason for this is that not much is known about this method for these types of reservoirs. So the objective of this thesis was to tackle the gap in knowledge about that. The case study was about the Volta Delta. At the moment, the delta is experiencing problems, and one of them is coastal erosion. By considering the Volta Delta as a case study, it was tried to answer the question of whether this concept could be a solution for the current problems at the coast.

When looking at the results and the conclusion, a distinction between two things should be made: the general concept and applying this concept with the corresponding social and economic consequences in Ghana. The concept itself seems to be worthy to investigate more in the future for the downstream part of a dam, especially when it is possible to transport sediment through the reservoir by means of other existing solutions. However, there is much research that needs to be done to understand how this concept works exactly and how the river responds to different operation options. It is better if these future researches could be executed with case studies where essential data are available so that the results would be reliable.

Because there is much research needed to understand this concept better, and because applying this method also depends on other practical matters such as the effects on power generation or the effects of the peak discharges on the floodplains, it is not likely that this method can be applied in the near future in Ghana. Considering the high rate of erosion along the coast at the moment, it is better for decision makers in the Volta Delta to focus on other (temporary) solutions for now. For other places outside Ghana (perhaps future dam locations), this method might be more useful to use if there are more details known about it.

A

DERIVATION OF RELEVANT FORMULAS

Below, the derivations of relevant formulas for sediment transport that are mentioned in chapter three are given. The text below is taken from the work of professor Van Rhee (TU Delft) [12].

A.1. SUSPENDED SEDIMENT: CONCENTRATION PROFILE

The sediment transport is determined with:

$$s = \rho_s \int_{z=0}^{z=H} c(z)u(z)dz \tag{A.1}$$

Where c = volume concentration, u = horizontal velocity of a particle and H = water depth. Hence to determine the suspended sediment transport both the velocity as the concentration distribution must be known. The concentration distribution follows from conservation of mass. Consider a (2D) control volume with length Δx and height Δz. For simplicity let us first consider only transport due to horizontal advection.

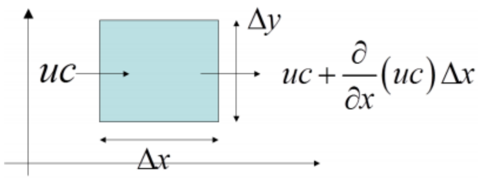

Figure A.1: Control volume with advection. From Van Rhee (2017) [12].

Transport by advection means that particles are transported by the average velocity in the flow. At the left (West) boundary the volume of particles carried by the flow reads $(uc)_W \Delta t \Delta z$, at the right (East) wall of the control volume the transport reads $(uc)_E \Delta t \Delta z$, where the subscript E or W denotes the West or East wall. The value of the product of velocity and concentration between the east and west boundary can be different, and subsequently, the volume of particles in the control volume will change. the change of volume of particles reads

$$\frac{\partial c}{\partial t} \Delta x \Delta z \Delta t \tag{A.2}$$

The relation between the fluxes at the east and west boundary reads:

$$(uc)_E = (uc)_W + \frac{\partial}{\partial x}(uc)\Delta x \tag{A.3}$$

With this result we can write:

$$\frac{\partial c}{\partial t}\Delta x\Delta z\Delta t = (uc)_W\Delta z\Delta t - (uc)_E\Delta z\Delta t = ((uc)_W - (uc)_W - \frac{\partial}{\partial x}(uc)\Delta x)\Delta z\Delta t \tag{A.4}$$

or:

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(uc) = 0 \tag{A.5}$$

Normally we are dealing with turbulent flow. In that case, a quantity is written as a time-averaged value plus a turbelent fluctuation:

$$u = \overline{u} + u' \tag{A.6}$$

$$c = \overline{c} + c' \tag{A.7}$$

Substitution in Eq. (6.9) leads to:

$$\frac{\partial(\overline{c} + c')}{\partial t} + \frac{\partial}{\partial x}((\overline{u} + u')(\overline{c} + c')) = 0 \tag{A.8}$$

or:

$$\frac{\partial \overline{c}}{\partial t} + \frac{\partial c'}{\partial t} + \frac{\partial}{\partial x}(\overline{uc} + \overline{u}c' + \overline{c}u' + u'c') = 0 \tag{A.9}$$

By time averaging three terms drop out since the time-averaged product of a fluctuation and an average value is zero. The equation reads now:

$$\frac{\partial \overline{c}}{\partial t} + \frac{\partial \overline{uc}}{\partial x} + \frac{\partial \overline{u'c'}}{\partial x} = 0 \tag{A.10}$$

the last term in the left-hand side of Eq. (3.10) is the transport due to turbulent fluctuations. In the momentum equation for turbulent flows a similar term is encountered. There the turbulent mixing results in a term $\rho\overline{u'w'}$. This momentum exchange term is modeled using the eddy-viscosity concept:

$$\rho\overline{u'w'} = -\rho\nu_e\frac{\partial u}{\partial z} \tag{A.11}$$

We can use the same concept here:

$$\overline{u'c'} = -\epsilon\frac{\partial c}{\partial x} \tag{A.12}$$

this is called the eddy diffusivity concept. The value of ϵ is closely related to the value of the eddy viscosity with the turbelent Schmidt-Prandtl number σ:

$$\epsilon = \frac{\nu_e}{\sigma} \tag{A.13}$$

Substitution of Eq. (3.12) in Eq. (3.10) yields the horizontal advection diffusion equation:

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(uc) = \frac{\partial}{\partial x}(\epsilon\frac{\partial c}{\partial x}) \tag{A.14}$$

In two dimensions with advection and diffusion the transport equation reads:

$$\frac{\partial c}{\partial t}+\frac{\partial}{\partial x}(uc)+\frac{\partial}{\partial z}(wc)=\frac{\partial}{\partial x}(\epsilon\frac{\partial c}{\partial x})+\frac{\partial}{\partial z}(\epsilon\frac{\partial c}{\partial z}) \tag{A.15}$$

For a special case an analytical solution of Eq. (3.15) is achieved. For a stationary uniform flow the derivatives to time t and coordinate x are zero and this equation simplifies to:

$$\frac{\partial}{\partial z}(wc)=\frac{\partial}{\partial z}(\epsilon\frac{\partial c}{\partial z}) \tag{A.16}$$

The vertical velocity of the particles is equal to the settling velocity w_s of the particles and the direction of this velocity is downward (now in negative z direction), so $w_s=-w$. So after substitution and integration the following equation is obtained:

$$w_s c=-\epsilon\frac{\partial c}{\partial z} \tag{A.17}$$

For a channel flow the eddy viscosity is often assumed to be a parabolic function:

$$v_e=ku_* z(1-\frac{z}{H}) \tag{A.18}$$

Where H = waterdepth, k = Von Karman constant (= 0.4), u_* is the friction velocity, per definition $u_*=\sqrt{\frac{\tau_b}{\rho}}$, where τ_b = bed shear stress. Substitution in Eq. (3.17) yields:

$$w_s c=-\frac{ku_*}{\sigma}z(1-\frac{z}{H})\frac{\partial c}{\partial z} \tag{A.19}$$

Rearranging and integration:

$$\int_{c_a}^{c}\frac{dc}{c}=-\frac{w_s\sigma}{ku_*}\int_{z_a}^{z}\frac{dz}{z(1-\frac{z}{H})} \tag{A.20}$$

The integral on the right side of Eq. (3.20) is a standard integral:

$$\int\frac{dx}{x(ax+b)}=-\frac{1}{b}ln\frac{ax+b}{x} \tag{A.21}$$

with $b=1$ and $a=-\frac{1}{H}$ and $x=z$ it follows that: or:

$$lnc-lnc_a=\frac{w_s\sigma}{ku_*}\left[ln\frac{1-\frac{z}{H}}{z}\right]_{z_a}^{z} \tag{A.22}$$

or:

$$lnc-lnc_a=\frac{w_s\sigma}{ku_*}\left(ln\frac{1-\frac{z}{H}}{z}-ln\frac{1-\frac{z_a}{H}}{z_a}\right) \tag{A.23}$$

Which can be writen as:

$$ln\frac{c}{c_a}=\frac{w_s\sigma}{ku_*}\left(ln\frac{H-z}{z}\cdot\frac{z_a}{H-z_a}\right) \tag{A.24}$$

Which finally yields:

$$\frac{c}{c_a}=\left(\frac{H-z}{z}\cdot\frac{z_a}{H-z_a}\right)^{\frac{w_s c}{ku_*}} \tag{A.25}$$

Which is the well known Rouse distribution. The exponent in the equation is called the Rouse number and is an important parameter to determine whether the transport mechanism is suspended transport or bed load. When $P = \frac{w_s \sigma}{k u_*}$ the concentration distribution is plotted for different values as a function of depth z/H in figure A.2. It is clear that for higher values of P, hence for high values of the settling velocity only a limited amount of sediment is suspended in the water column. In literature, a value of $P = 2{,}5$ is often referred to as the limit of suspended transport.

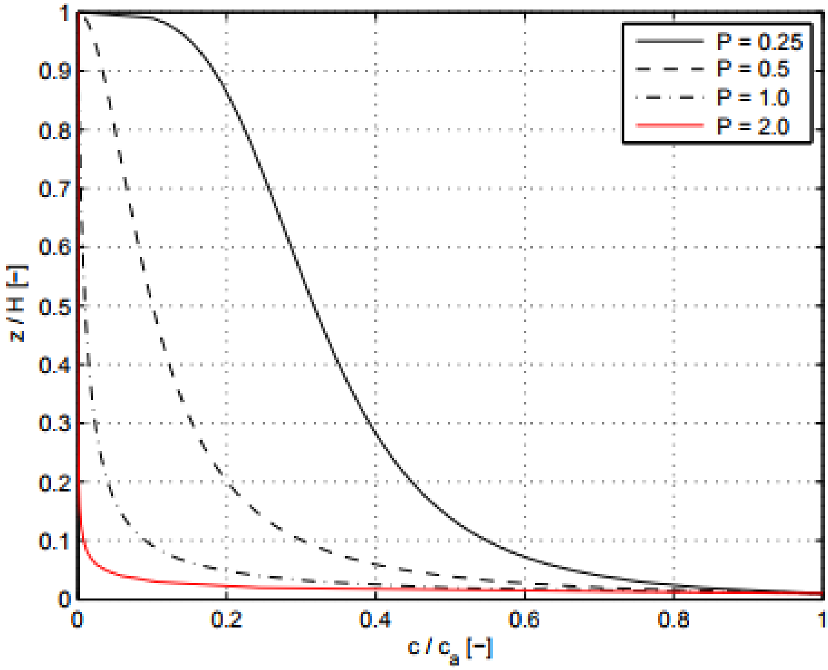

Figure A.2: Concentration distribution. From Van Rhee (2017) [12].

A.2. SUSPENDED SEDIMENT: SETTLING VELOCITY OF NON-COHESIVE MATERIAL

For one-dimensional settling, the particle velocity v_p is computed with:

$$(V_p\rho_s + M_a)\frac{dv_p}{dt} = A_p C_D \frac{1}{2}\rho_w \left|v_w - v_p\right|(v_w - v_p) + V_p g(\rho_s - \rho_w) \tag{A.26}$$

Where V_p = volume of particle, ρ_s = density of particle, ρ_w = density of water, v_p = velocity of particle, v_w = velocity of water surrounding particle, M_a = added mass coefficient, A_p = surface area of particle in flow direction. The added mass is determined with:

$$M_a = C_m \rho_w V_p \tag{A.27}$$

For a stationary situation ($\frac{dv_p}{dt} = 0$) and stagnant flow conditions ($v_w = 0$) this expression reads:

$$v_p = \sqrt{\frac{2\Delta g V_p}{C_D A_p}} \tag{A.28}$$

Where the specific density Δ is defined as:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{A.29}$$

For a sphere with diameter d, this reduces to the well known general equation for the settling velocity for a single sphere:

$$w_0 = \sqrt{\frac{4\Delta g d}{3C_D}} \tag{A.30}$$

The drag coefficient C_D is a function of the particle Reynolds number defined as $Re_p = \frac{w_0 d}{v}$. Three different regimes can be distinguished: the laminar, turbulent and transition regime. For each regime a different relation applies:

$$Re_p < 1 C_D = \frac{24}{Re_p} \tag{A.31}$$

$$1 < Re_p < 2000 C_D = \frac{24}{Re_p} + \frac{3}{\sqrt{Re_p}} + 0,34 \tag{A.32}$$

$$Re_p > 2000 C_D = 0,4 \tag{A.33}$$

For the laminar regime, the relation for the drag coefficient can be substituted in Eq. (3.30) which yields the so-called Stokes equation for the settling velocity:

$$w_0 = \frac{\Delta g d^2}{18 v} \tag{A.34}$$

For the turbelent regime the value for C_D is constant which results in the following equation:

$$w_0 = 1.8\sqrt{\Delta g d} \tag{A.35}$$

In the transition regime, the value for the settling velocity is found by iteration of the C_D value of application of empirical equations. Often used is the following equation of Ruby and Zanke:

$$w_0 = \frac{10 v}{D}\left(\sqrt{1 + \frac{\Delta g d^3}{100 v^2}} - 1\right) \tag{A.36}$$

More recently the following equation is published that can be used to compute the settling velocity directly over a very wide range [50]:

$$w_0 = \frac{\Delta g d^2}{C_1 \nu + \sqrt{0.75 C_2 \Delta g d^3}} \tag{A.37}$$

Where $C_1 = 18$ and $C_2 = 1$ for natural sands and $C_2 = 0.44$ for spheres.

Equation (3.30) determines the settling velocity of a single particle. When a large number of particles is settling in a confined space, the settling velocity of the individual particles is reduced. The influence of the volume concentration on the settling velocity of a monosized mixture (mixture with particles of the same size) is written as [29]:

$$w_s = w_0 (1-c)^n \tag{A.38}$$

Where w_0 = the settling velocity of a single particle and c = the volume concentration. Figure A.3 shows the influence of volume concentration on the settling velocity. The velocity is normalized with the settling velocity w_0. Relations for two different particle diameter is shown. The reduction for coarse sand is less because of the lower value of the exponent n. the exponent n is a function of the particles Reynolds number defined as $Re_p = \frac{w_0 d}{\nu}$ and varies between 2,4 (coarse particles) and 4,65 (fine particles). According to Van Wijk et al. (2012) [51] for coarse gravel this equation still holds and a value of n =2,4 was found with fluidization experiments. A convenient way to compute the value of the value of n is the method of Rowe (1987) [30], which is a smooth representation of the original relations of Richardson and Zaki (1954) [29]:

$$n = \frac{4,7 + 0,41 Re_p^{0,75}}{1 + 0,175 Re_p^{0,75}} \tag{A.39}$$

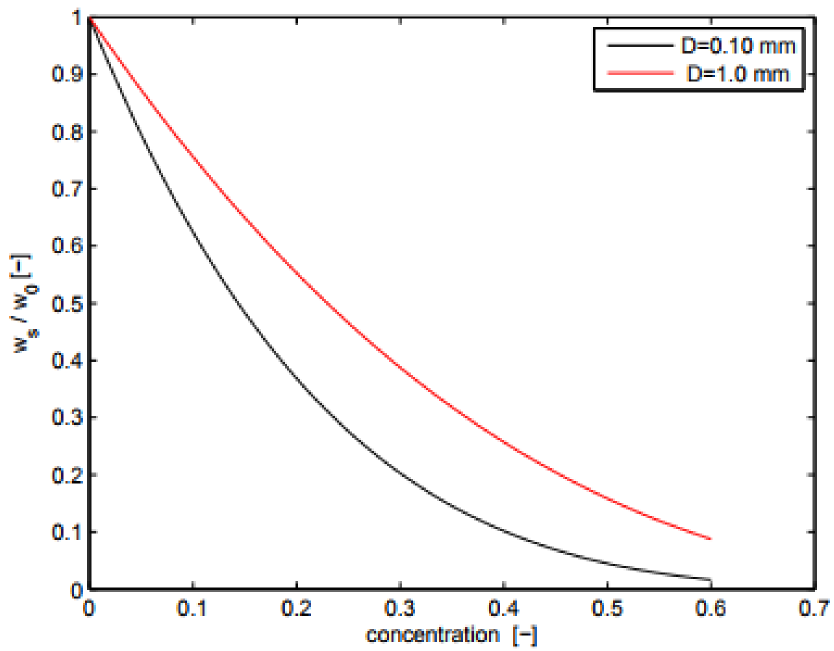

Figure A.3: Settling velocity as a function of the concentration. From Van Rhee (2017) [12].

A.3. SUSPENDED SEDIMENT: EROSION AND DEPOSITION FLUXES OF NON-COHESIVE MATERIAL

In a situation where erosion and deposition take place, the sedimentation velocity can be expressed by:

$$v_{sed} = \frac{S - E}{\rho_s(1 - n_0 - c)} \tag{A.40}$$

Where S = sedimentation flux defined as:

$$S = \rho_s w_s c = \rho_s w_0 c(1-c)^n \tag{A.41}$$

And E is the erosion or pick-up flux. It is calculated with empirical expressions and often presented in a dimensionless shape using Φ_p:

$$\Phi_p = \frac{E}{\rho_s \sqrt{g\Delta d}} \tag{A.42}$$

A well known equation is van Rijn (1984):

$$\Phi_p = 0{,}00033 D_*^{0.3} \left(\frac{\theta - \theta_{cr}}{\theta_{cr}}\right)^{1.5} \tag{A.43}$$

Where D_* is defined as:

$$D_* = d\sqrt[3]{\frac{\Delta g}{v^2}} \tag{A.44}$$

The dimensionless bed shear stress θ is the Shields parameter. Movement of particles will take place when the actual shear stress is larger than the critical value, hence when $\theta > \theta_{cr}$. Numerous function exist to determine the critical Shields value. A convenient relation is (Brownlie, 1981) [49]:

$$\theta_{cr} = 0{,}22 R_p^{-0{,}6} + 0{,}06 \exp(-17{,}77 R_p^{-0{,}6}) \tag{A.45}$$

Where R_p = particle Reynolds number but not with the particle settling velocity as velocity scale, but defined as:

$$R_p = \frac{d\sqrt{\Delta g d}}{v} \tag{A.46}$$

B
The peak duration and its influence on the river

In chapter six, the results of different scenario sets were shown. The set with changing peak durations showed the most interesting results. According to the model, the duration of a peak does not have much influence on the amount of transported sediment. In this appendix, it is tried to figure out what a possible cause for this behavior might be. It is written in the same order as of how the search for an answer was conducted (after making sure there was no error made in the input data).

B.1. Is this behavior site specific?

In the results, all the plots for the sediment transport were shown for one specific part at the river near the ocean. The main reason for this is that the focus of this thesis was on sediment transport towards to coast. The first question that was investigated was whether the same behavior could be noticed in other parts of the river. To check that, a plot is made for the width averaged sediment transport of sand for a peak of 2, 4 and 6 weeks. In figure B.1 it can be seen that the sediment mean transport rate is almost the same everywhere at the river. So it can be said that the observed behavior is not site specific.

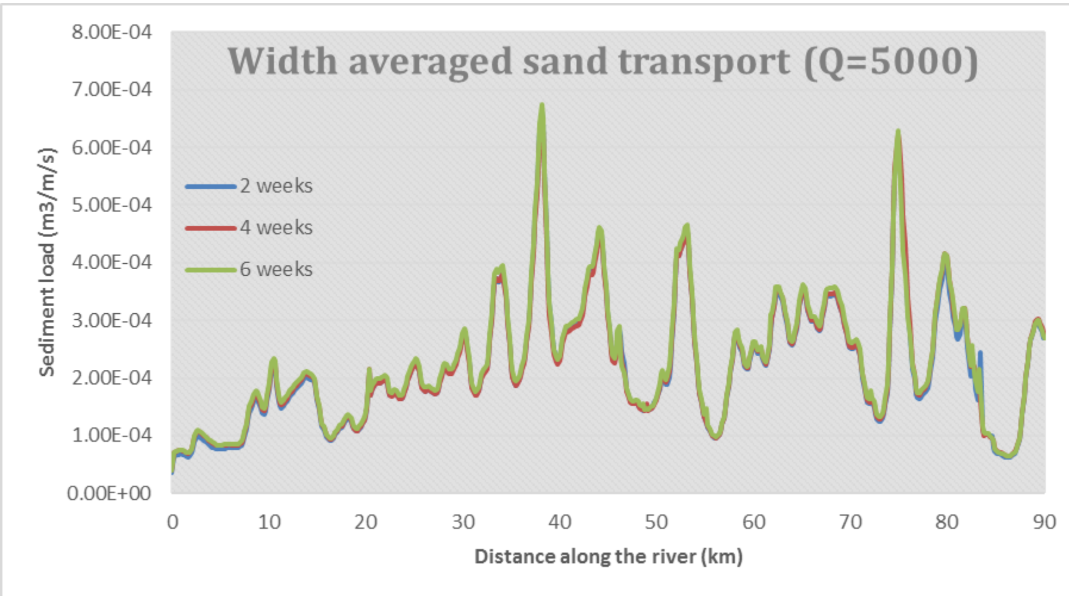

Figure B.1: Mean width averaged sand transport along the river

B.2. IS IT BECAUSE OF THE MEAN VALUE?

One idea about the results is that the actual behavior of the river is not visible anymore because the plotted value was the mean sediment load after a simulation of 10 years. So it might be possible that the transport load of the 6-weeks scenario is higher in the first years compared with the 2-weeks scenario and that both scenarios reach the same transport rate after different years within 10 years. To check that, the averaged sand load for each year for both scenarios are plotted. In figure B.2 it can be seen that the transport rate is almost the same for each year for both scenarios. It can also be seen that the transport rate decreases in time.

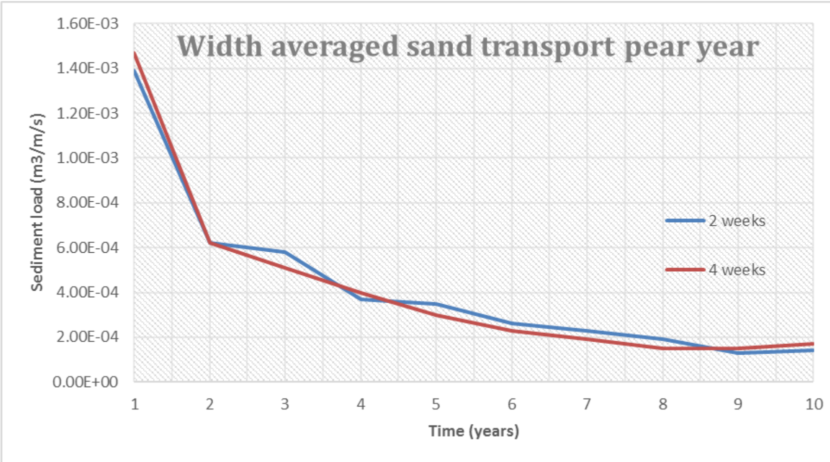

Figure B.2: Width averaged sand transport per year near the coast

B.3. LOOKING AT THE RIVER BEHAVIOR DURING THE YEAR

Since the observed behavior is the same for all river parts and each year, it might be interesting to see what happens during one year or one discharge peak. Figure B.3 shows the transport rate of sand in time between July and October. During the other months, the transport rate is negligible compared with this period. This figure shows how it is possible to transport the equal amount of sediment during a year with two different discharge regimes. Apparently, the operation regime with a peak discharge of two weeks has a higher transport rate in the rising part of the peak. It also has a higher peak value for the transport rate. The other scenario, on the other hand, has a lower peak but a longer duration. Apparently, the two different shapes are responsible for the same amount of sediment transport each year.

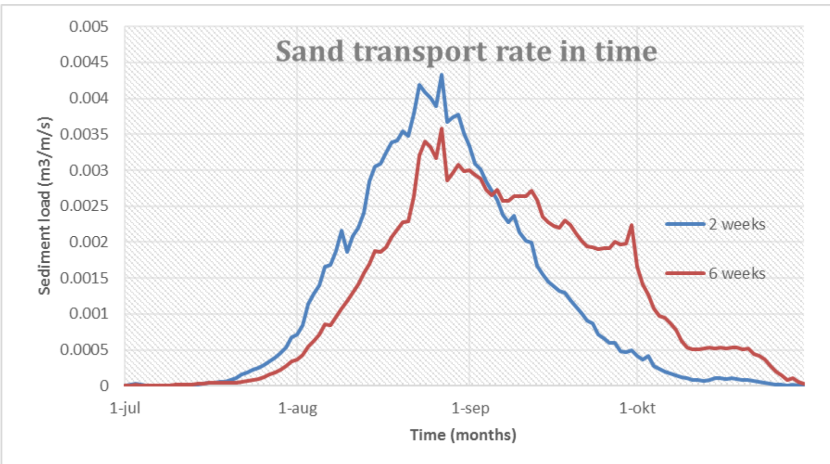

Figure B.3: Transport rate of sand between July and October for two operation scenarios

B.4. MORE DETAILS ABOUT THE SHAPE OF THE TRANSPORT RATE CURVE

The previous figure shows that the different transport rate curves can produce the same amount of sediment in the same year. But this brings up the next question: why do the shapes look like that? To get some insight about the processes that shape that transport rate curve, the behavior of the river in the first year is investigated more closely. Figure B.4 shows the water depth during a discharge peak for both operation scenarios. This pattern follows from the discharge pattern in time for the two scenarios (figure B.5). When the bed level and the width are the same, the water depth will be the same for both situations. For the 6-weeks simulation, the discharge stays larger for a longer time and because of that the water level will rise higher than that of the 2-weeks simulation.

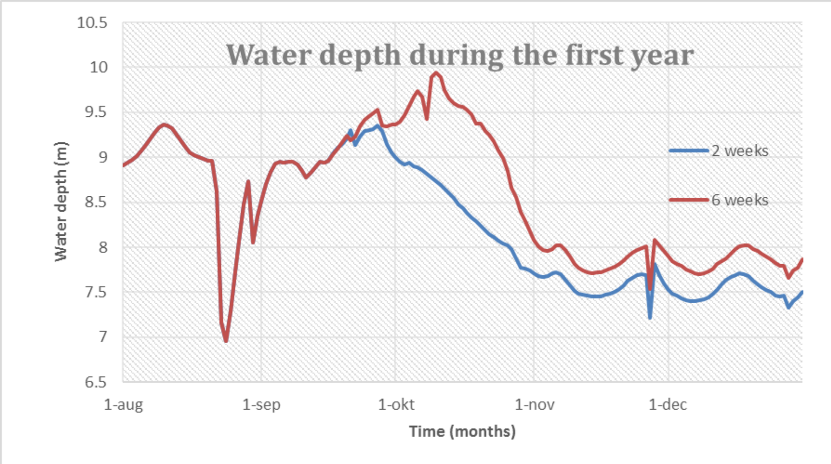

Figure B.4: Water depth in time for two different operation scenarios

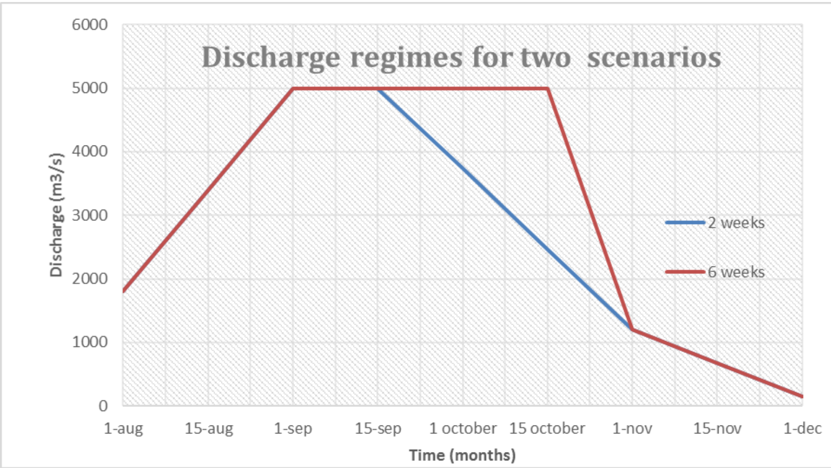

Figure B.5: Discharge scheme for two different operation scenarios

Since the discharge divided by the cross-sectional area is equal to the flow velocity ($U = Q/B\dot{d}$), the flow velocity of the 6-weeks scenario will be higher than that of the 2-week scenario. This can be seen in figure B.6. A higher velocity corresponds with a higher bed shear stress which can be seen in figure B.7. With a higher bed shear stress, more material can be eroded from the bed which means a higher suspended sediment concentration near the bottom (figure B.8). A higher concentration and flow velocity result in a higher sediment transport (figure B.9). Since there are no incoming sediments, the only transported sediments will be taken from the bottom. This means that there will be more erosion in the bed of the 6-weeks scenario compared with the other scenario (figure B.10).

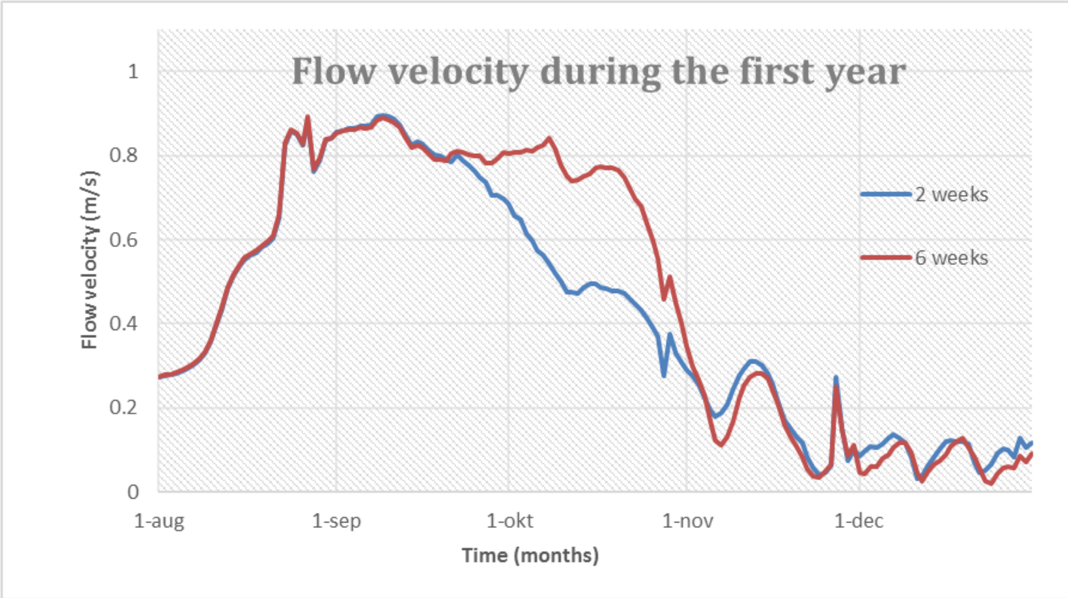

Figure B.6: Flow velocity in time for different operation scenarios

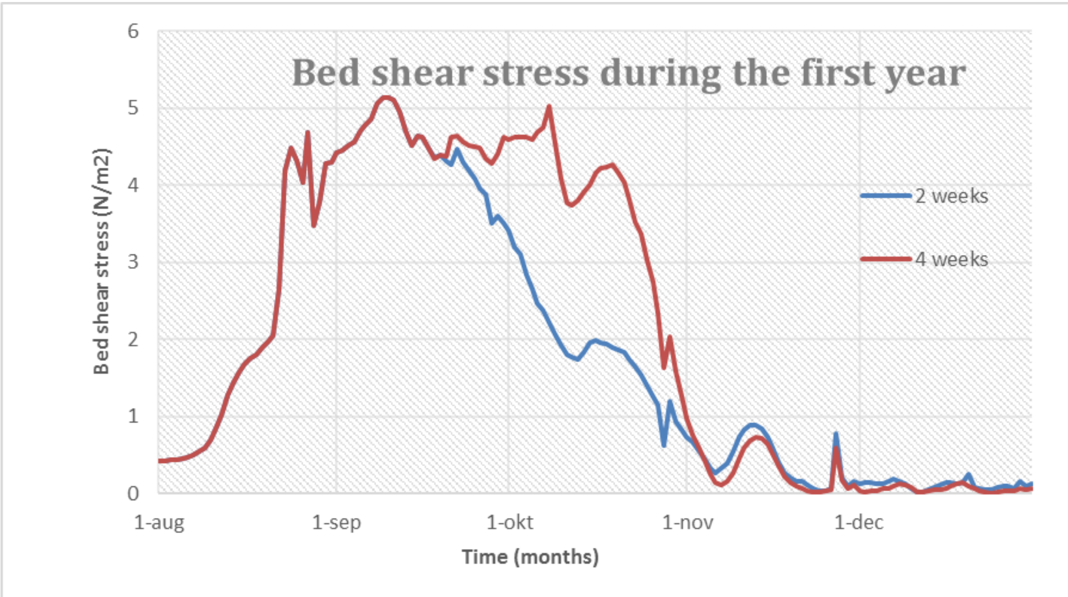

Figure B.7: Bed shear stress in time for different operation scenarios

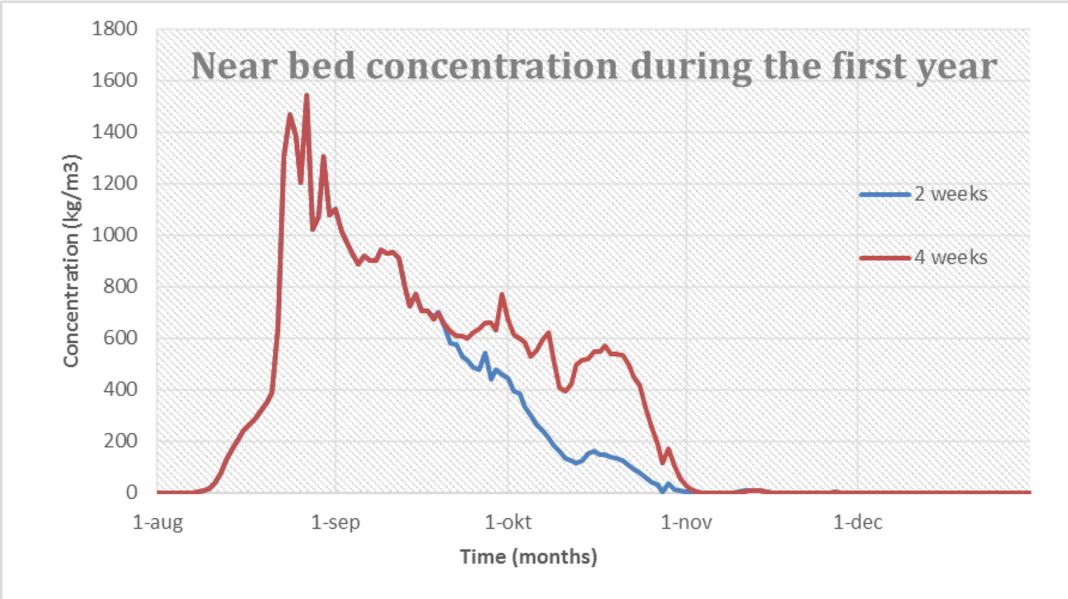

Figure B.8: Concentration in time for different operation scenarios

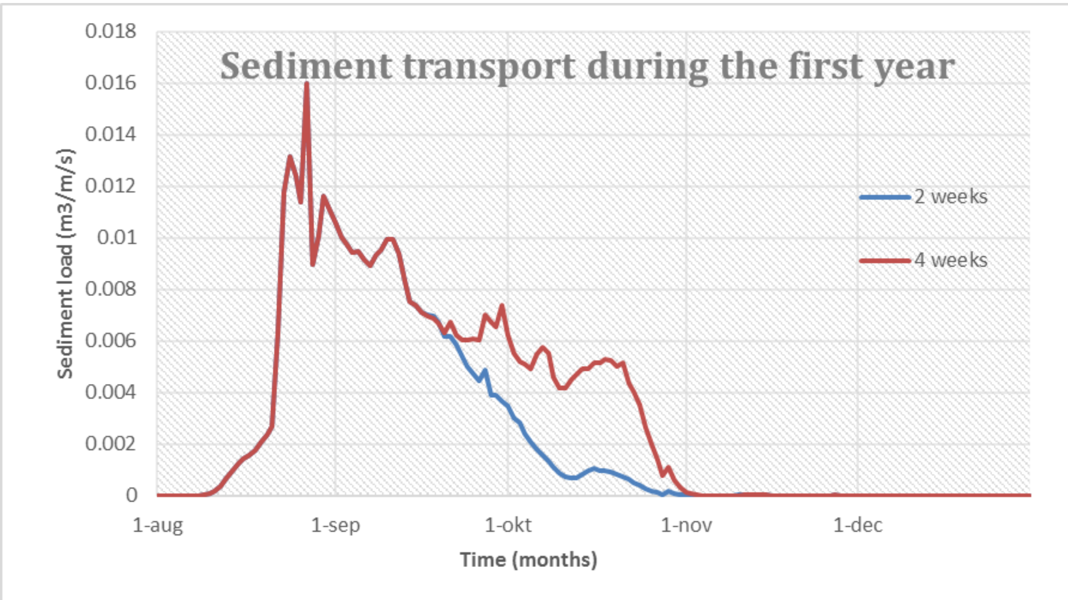

Figure B.9: Sediment transport in time for different operation scenarios

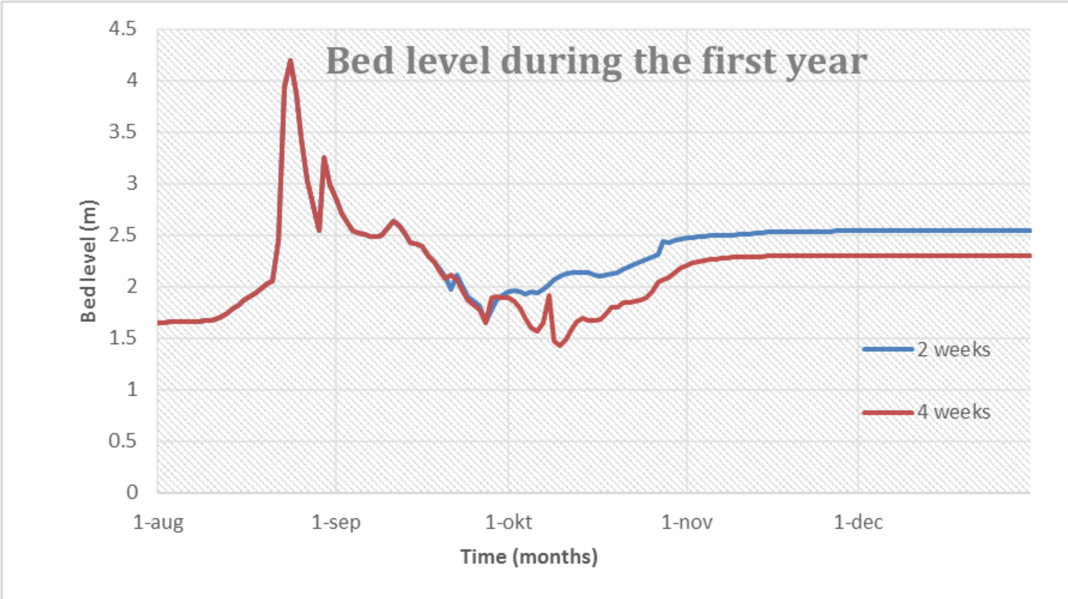

Figure B.10: Bed level in time for different operation scenarios

In the second year, similar behavior occurs. The difference with the first year, however, is that the water depths in the two situations before the discharge peak are not equal. The 6-weeks scenario has a higher water depth. If it can be assumed that the width stays the same for both situations, and the discharge is also the same until mid-September, then the flow velocity for the 2-weeks scenario will be higher until then. After that, the discharge for the 6-weeks scenario will be much higher compared with the other scenario, and thus the 6-weeks scenario will have a higher flow velocity during the decreasing part of the discharge peak. This result can be seen in figure B.11 As for the first year, the bed shear stress and concentration will follow the same pattern, and this will lead to a sediment transport pattern as can be seen in figure B.12 The same behavior can start again in the third year leading to the same amount of transported sediment.

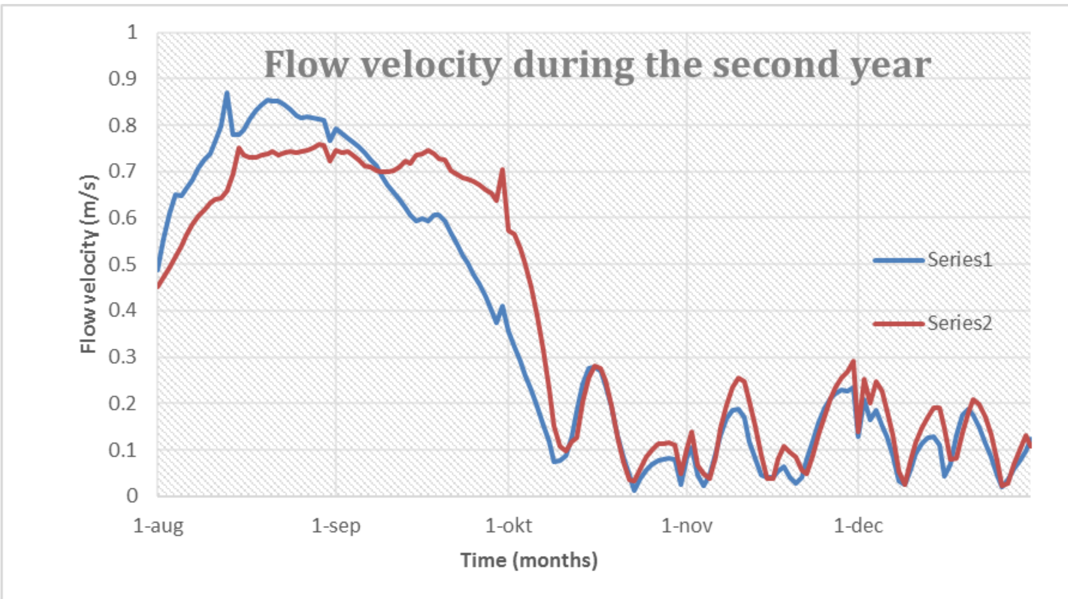

Figure B.11: Flow velocity in time for different operation scenarios

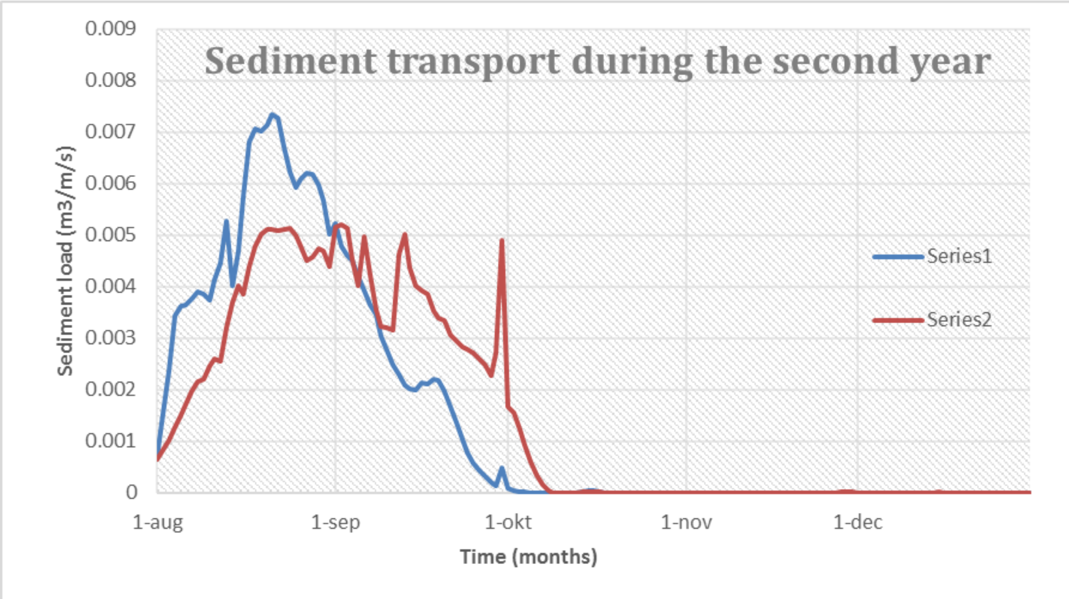

Figure B.12: Sediment transport in time for different operation scenarios

B.5. CONCLUSION AND REMARKS

It is known that the discharge influences sediment transport and bed erosion. On the other hand, a changed bed level will also change the impact of the discharge later. There is continuous feedback between the involved processes in the river. This feedback might tend to change the bed level in such a way that the corresponding eroded and the transported amount is more or less the same despite changes in time.

It should be mentioned that in the simulations that were made, no sediments entered the river from upstream where the dam is located. All transported material is originating from the river bed. In a situation where sediments are entering the river, this behavior might not be observed because if there is sediment available from upstream and that can be transported without interacting with the river bed.

Furthermore, it should be mentioned that in the given example for the first two years of the situation, a constant bed level and a non-changing width is assumed. In reality, this is not the case. However, the results still show no change in the amount of transported sediments downstream. Apparently, not only the bed level but the whole cross-section of the river adjust in such a way that it produces the same amount of sediment load. This, however, should be researched in more details because it was not looked at in this thesis.

C

DATA MODEL PARAMETERS

In this appendix, parameters that will be needed for the models are listed with their sources and provided with comments where it is needed.

C.1. VOLTA RIVER DISCHARGE

Before construction of the Akosombo Dam, the discharge of the Volta River was variable with high flows during the wet seasons. Table C.1 shows monthly average values measured at Senchi before the Akosombo Dam was built.

Year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Average
1936	116	62	34	37	207	326	355	1020	3130	5215	876	255	**969**
1937	103	52	35	45	92	201	538	981	4675	5264	695	127	**1067**
1938	69	30	21	16	108	259	686	981	3209	3589	956	217	**845**
1939	118	51	44	52	129	286	959	1749	5092	5822	1045	196	**1295**
1940	116	62	34	37	85	240	221	1317	3937	3935	1605	193	**982**
1941	102	44	21	22	129	463	694	2655	6814	3463	353	170	**1244**
1942	97	29	18	18	195	466	471	761	2846	1052	334	105	**533**
1943	28	13	9	33	142	233	379	864	3040	3746	1084	248	**818**
1944	116	55	65	69	110	108	377	1697	4839	3779	491	143	**987**
1945	76	11	6	17	40	128	691	2716	6819	6745	879	159	**1524**
1946	52	18	18	38	48	85	180	541	2011	4309	1724	187	**768**
1947	107	76	47	19	70	205	644	2742	8683	6474	835	191	**1674**
1948	106	38	45	12	131	441	633	1039	4672	3005	331	91	**879**
1949	28	10	9	22	28	292	982	3631	9005	4470	803	172	**1621**
1950	70	35	12	16	72	121	357	962	2356	2486	432	101	**585**
1951	38	12	12	21	96	146	498	1254	5042	8106	5884	435	**1795**
1952	153	78	48	30	80	218	782	1361	4671	9350	2023	260	**1588**
1953	153	91	80	42	51	1223	1978	3027	4926	5343	779	158	**1488**
1954	80	46	33	52	112	496	547	679	3508	4511	1159	234	**955**
1955	129	78	80	80	129	328	1776	4242	7113	7793	1998	289	**2003**
1956	153	90	80	93	80	206	247	411	2774	3714	446	163	**705**
1957	93	43	24	37	134	1398	2196	2694	6751	8618	2800	638	**2119**
1958	181	90	61	56	125	247	178	281	1703	1402	292	173	**399**
1959	101	54	37	64	155	201	866	1011	4047	4535	588	173	**986**
1960	70	24	10	51	38	208	737	1955	4888	6723	878	181	**1314**
1961	93	27	8	12	47	122	1098	1695	2493	2925	317	133	**748**
1962	76	26	17	34	185	635	1850	2642	6476	6979	1513	365	**1733**
1963	115	63	83	76	162	253	2605	5822	12501	10712	3821	374	**3049**
1964	168	85	44	75	109	323	574	2410	8051	6505	938	306	**1632**
1965	236	123	57	45	69	468	2281	3011	5481	3424	704	301	**1350**
1966	142	51	29	63	182	327	419	2598	4798	4977	1130	300	**1251**
Average	**106**	**51**	**36**	**41**	**108**	**344**	**864**	**1895**	**5044**	**5128**	**1217**	**227**	1255

Table C.1: Monthly averaged discharge at Senchi (cubic meter per second) from 1936 to 1966. Source: Volta River Authority.

The discharge of the Volta River is currently more or less constant because of the Akosombo Dam. Table C.2 contains average monthly plant discharge values of the Akosombo Dam from 1965 to 2012. Those values are obtained from daily discharge data received from the Volta River Authority. Table C.3 contains the same values for the Kpong Dam. This dam acts as a run-of-the-river dam, so the discharges released from the Kpong are more or less the same as the releases in Akosombo (differences no larger than 5%).

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
1965	1355	1213	1203	1187	1168	1192	1196	1343	1441	1410	1350	1019	1256
1966	154	156	154	172	157	160	165	162	160	160	163	168	**161**
1967	174	195	228	268	303	372	397	419	405	437	501	512	**351**
1968	527	524	524	527	529	525	525	536	543	540	535	537	**531**
1969	542	552	560	565	568	569	572	566	571	573	580	574	**566**
1970	584	596	586	595	597	594	597	595	594	592	592	592	**593**
1971	602	605	610	611	615	622	622	627	609	562	575	577	**603**
1972	577	594	598	616	623	707	736	743	743	746	756	752	**683**
1973	772	787	739	791	810	813	817	813	802	814	824	832	**801**
1974	843	862	862	857	871	865	874	865	851	834	851	836	**856**
1975	852	876	852	862	826	807	792	788	782	777	803	835	**821**
1976	853	864	873	865	864	865	857	865	870	874	880	880	**868**
1977	874	895	952	976	1028	1017	699	806	895	942	1002	1003	**924**
1978	1025	996	995	980	497	496	571	662	728	814	887	931	**799**
1979	958	878	973	941	999	983	971	1005	999	1015	1048	1058	**986**
1980	1083	1029	1120	1087	1129	1115	1118	1106	1134	1025	1086	1106	**1095**
1981	1112	1143	1145	1147	1149	1178	1166	1151	1155	1119	1033	974	**1123**
1982	961	976	975	976	967	896	965	984	978	952	735	735	**925**
1983	718	656	634	603	603	461	367	379	381	414	388	284	**491**
1984	277	891	251	267	906	924	900	875	831	780	818	829	**712**
1985	477	512	522	522	599	666	565	554	444	543	650	651	**559**
1986	690	697	714	713	750	802	882	895	897	893	865	818	**801**
1987	832	891	922	900	906	924	900	875	831	780	818	829	**867**
1988	832	883	904	954	975	983	904	864	760	772	866	887	**882**
1989	900	949	976	1013	1010	1015	1000	939	800	798	893	965	**938**
1990	992	994	1001	968	981	991	954	979	984	1008	993	1032	**990**
1991	1075	1118	1117	1110	1121	1103	1058	1014	1011	1009	1070	1087	**1075**
1992	1109	1135	1175	1179	1147	1172	1160	1120	1115	1104	1130	1147	**1141**
1993	1076	1119	1117	1110	1121	1104	1060	1014	1011	1009	1069	1087	**1075**
1994	1299	1308	1282	1250	1212	1181	1188	1116	1059	921	954	1133	**1159**
1995	1148	1182	1171	1120	1118	1114	1124	1137	1119	1113	1183	1190	**1143**
1996	1245	1270	1279	1247	1244	1237	1242	1241	1188	1150	1257	1242	**1237**
1997	1327	1353	1295	1309	1299	1291	1280	1267	1310	1320	1336	1299	**1307**
1998	1081	835	646	531	519	491	529	586	700	896	1004	1092	**742**
1999	1093	988	936	895	823	964	1005	1085	868	992	1094	1121	**989**
2000	1114	1068	1118	1149	1124	1141	1077	1343	1405	1371	1451	1394	**1230**
2001	1355	1213	1203	1187	1168	1192	1196	1343	1441	1410	1350	1019	**1256**
2002	1036	1085	912	866	877	903	937	940	1009	898	926	948	**945**
2003	752	820	733	683	593	552	656	736	740	753	750	834	**717**
2004	834	872	916	936	926	1031	1046	964	894	918	1015	1031	**949**
2005	1026	1129	961	1024	1059	1032	871	870	955	963	1121	1155	**1014**
2006	1164	1096	1142	1093	1186	1156	1137	1095	811	792	887	981	**1045**
2007	1046	1120	806	541	529	496	474	479	503	697	835	719	**687**
2008	774	1292	1163	1097	1117	1082	941	1111	1142	1306	1293	1212	**1127**
2009	1131	1216	1184	1209	1250	1139	1223	1103	1067	1136	1267	1329	**1188**
2010	1364	1275	1290	1342	1334	1186	1162	1035	874	1147	1279	1145	**1203**
2011	1092	1184	1270	1280	1296	1312	1232	1320	1372	1333	1283	1316	**1274**
2012	1408	1455	1413	1386	1390	1348	1309	1258	1404	1369	1443	1335	**1376**
Average	**919**	**945**	**917**	**906**	**914**	**912**	**896**	**908**	**900**	**912**	**948**	**938**	**918**

Table C.2: Monthly averaged plant discharge values (cubic meter per second) for the Akosombo Dam from 1965 to 2012. The last column contains average discharge values for each year. The last row contains averaged values over the years per month. Source: Volta River Authority.

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
1985	477	525	540	542	610	685	603	580	467	567	669	651	**576**
1986	688	701	702	704	743	792	869	880	895	879	855	805	**793**
1987	817	894	919	898	907	912	899	882	835	821	845	848	**873**
1988	830	882	897	938	950	986	918	879	790	796	876	894	**886**
1989	889	925	948	994	1000	1003	988	947	818	810	896	948	931
1990	978	979	987	961	971	973	945	967	978	1004	977	1023	**979**
1991	978	979	987	1092	1079	1008	1088	1035	1023	1023	1075	1072	**1037**
1992	1025	1034	1068	1091	1076	1006	1088	1035	1023	1022	1075	1072	**1051**
1993	1025	1034	1068	1091	1076	1006	1088	1035	1023	1022	1075	1072	**1051**
1994	966	1072	1246	1238	1207	1177	1165	1084	1043	917	951	1113	**1098**
1995	1117	1146	1128	1085	1087	1097	1114	1129	1111	1091	1159	1158	**1118**
1996	1211	1245	1258	1217	1212	1210	1231	1227	1165	1132	1224	1221	**1213**
1997	1275	1298	1233	1279	1267	1239	1245	1241	1288	1277	1320	1252	**1268**
1998	1082	827	637	530	515	502	532	590	698	894	990	1076	**739**
1999	1092	969	900	908	836	951	1013	1078	891	982	1056	1064	**978**
2000	1073	1032	1077	1089	1075	1063	1053	1278	1313	1274	1340	1280	**1162**
2001	1285	1190	1160	1158	1142	1180	1171	1289	1332	1290	1249	997	**1204**
2002	1015	1059	918	888	897	907	955	965	1020	893	931	944	**949**
2003	764	834	752	731	624	620	689	768	769	775	786	839	**746**
2004	855	885	934	945	954	1007	1033	974	902	926	1009	1009	**953**
2005	993	1054	947	1002	1046	1025	882	874	957	994	1091	1122	**999**
2006	1127	1073	1090	1017	1062	1124	1137	1093	836	847	903	978	**1024**
2007	1003	1086	810	560	555	505	498	487	518	698	847	728	**691**
2008	818	935	1024	1009	992	972	945	1082	1103	1216	1211	1151	**1038**
2009	1089	1159	1131	1157	1180	1193	1166	1088	1041	1083	1169	1240	**1141**
2010	1283	1208	1214	1238	1226	1120	1098	1021	893	1094	1222	1100	**1143**
2011	1067	1084	1097	1075	1150	1229	1174	1241	1291	1243	1210	1213	**1173**
2012	1275	1326	1185	1278	1223	1279	1241	1184	1290	1110	1176	1223	**1232**
Average	**1003**	**1016**	**995**	**990**	**988**	**992**	**994**	**998**	**975**	**989**	**1042**	**1039**	1002

Table C.3: Monthly averaged plant discharge values (cubic meter per second) for the Kpong Dam from 1985 to 2012. The last column contains average discharge values of each year. The last row contains averaged values over the years per month. Source: Volta River Authority.

C.2. WATER LEVELS LAKE VOLTA

In table C.4 the monthly averaged values for the water levels at the Volta Lake are given. In one year, there is a difference of approximately 2 meters. The lowest water level ever measured (after filling up the reservoir) is $71,70m$, and the highest water level is $84,49m$

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
1965	60,36	60,32	60,26	60,21	60,25	60,44	62,47	65,11	68,27	71,32	72,24	72,22	64,45
1966	72,08	71,89	71,70	71,56	71,48	71,49	71,87	72,63	74,84	77,02	78,03	78,04	73,55
1967	77,71	77,70	77,52	77,39	77,28	77,21	77,32	77,70	79,05	81,12	81,69	81,53	78,60
1968	81,26	80,97	80,68	80,44	80,26	80,35	80,95	82,17	82,75	83,02	83,00	83,10	81,58
1969	82,88	82,45	81,99	81,48	81,07	80,88	80,88	81,31	82,47	83,70	83,68	83,33	82,18
1970	82,96	82,50	82,10	81,68	81,23	80,94	80,71	80,78	82,14	83,90	83,57	83,19	82,14
1971	82,94	82,55	82,11	81,45	81,04	80,81	80,84	81,40	82,44	83,63	83,70	83,45	82,20
1972	83,16	82,86	82,54	82,23	81,67	81,61	81,53	81,59	82,12	82,79	82,96	82,65	82,31
1973	82,31	81,92	81,53	81,15	80,80	80,52	80,41	80,72	81,95	82,92	82,90	82,51	81,64
1974	82,13	81,70	81,28	80,88	80,52	80,20	80,02	80,44	81,82	83,83	84,16	83,95	81,74
1975	83,47	83,07	82,67	82,35	82,05	81,77	81,74	82,22	82,74	83,81	83,90	83,61	82,78
1976	83,23	82,83	82,44	82,05	81,72	81,49	81,42	81,31	81,33	81,69	82,50	82,42	82,04
1977	82,05	81,71	81,20	80,71	80,26	79,86	79,52	79,33	79,86	80,87	80,78	80,34	80,54
1978	79,83	79,77	78,82	78,34	78,05	78,02	78,03	78,29	78,72	79,35	79,63	79,27	78,84
1979	78,84	78,51	77,84	77,33	76,88	76,59	77,07	78,48	80,13	81,92	82,19	81,93	78,97
1980	81,32	80,81	80,13	79,59	79,10	78,75	78,37	78,52	80,00	81,47	81,66	81,26	80,08
1981	80,72	80,20	79,68	79,15	78,69	78,21	77,96	78,28	79,00	79,59	79,41	78,95	79,15
1982	78,47	77,94	77,44	76,91	76,44	76,02	75,72	75,46	75,71	76,08	75,92	75,52	76,47
1983	75,05	74,54	74,10	73,68	73,28	73,06	73,04	73,07	73,32	73,65	73,41	73,15	73,61
1984	72,90	76,33	75,80	75,24	74,69	74,23	73,90	74,17	76,34	78,18	78,34	77,93	75,67
1985	74,91	74,56	74,19	73,85	73,48	75,06	73,22	74,42	76,83	78,88	79,05	78,75	75,60
1986	78,35	77,93	77,56	77,18	76,80	76,40	76,12	76,03	76,67	77,64	77,70	77,32	77,14
1987	76,83	76,33	75,80	75,24	74,69	74,23	73,90	74,17	76,34	78,18	78,34	77,93	76,00
1988	77,44	76,94	76,42	75,89	75,33	74,88	74,82	75,31	76,95	79,22	79,36	78,97	76,79
1989	78,47	77,97	77,46	76,91	76,41	75,91	76,07	76,81	79,73	83,01	83,53	83,18	78,79
1990	82,77	82,29	81,81	81,31	80,93	80,53	80,30	80,32	80,67	80,92	80,63	80,19	81,06
1991	79,70	79,14	78,55	78,03	77,67	77,84	78,32	79,79	82,10	83,61	83,78	83,40	80,16
1992	82,91	82,60	81,85	81,35	80,90	80,52	80,27	80,10	80,23	80,68	80,37	79,87	80,97
1993	79,26	78,65	78,05	77,42	76,88	76,37	75,95	76,16	77,58	78,69	78,62	78,09	77,64
1994	77,40	76,69	75,97	75,27	74,59	74,02	73,46	73,13	74,33	76,88	78,10	77,77	75,63
1995	77,18	76,58	75,92	75,37	74,90	74,42	74,16	74,92	77,01	78,72	78,99	78,48	76,39
1996	77,89	77,23	76,56	75,86	75,24	74,82	74,84	74,86	76,20	78,18	78,53	77,94	76,51
1997	77,31	76,61	75,85	75,17	74,52	73,96	73,65	73,47	74,21	75,59	75,57	74,90	75,07
1998	74,19	73,58	73,08	72,61	72,36	72,25	72,36	72,70	73,83	75,96	76,72	76,24	73,82
1999	75,65	75,09	74,56	74,08	73,64	73,23	73,02	73,29	75,47	79,16	80,37	80,01	75,63
2000	79,50	79,00	78,44	75,42	77,43	77,01	76,95	77,21	78,39	79,95	80,01	79,43	78,23
2001	78,76	78,14	77,47	76,86	76,34	75,79	75,38	75,12	75,54	76,68	73,86	75,71	76,30
2002	75,13	74,49	73,90	73,36	72,93	72,55	72,37	72,67	73,81	74,80	75,14	74,66	73,82
2003	74,12	73,62	73,09	72,65	72,28	72,11	72,41	72,94	74,72	77,31	78,00	77,68	74,24
2004	77,28	76,85	76,32	75,83	75,43	75,08	74,79	75,24	76,73	78,07	78,14	77,72	76,46
2005	77,19	76,58	76,06	75,53	75,05	74,61	74,44	74,85	75,60	76,79	77,08	76,52	75,86
2006	75,89	75,30	74,66	73,94	73,32	72,88	72,48	72,19	72,68	74,31	75,03	74,62	73,94
2007	73,97	73,30	72,64	72,18	71,95	71,85	71,68	72,20	75,05	77,83	78,09	77,81	74,04
2008	77,40	76,89	76,26	75,68	75,21	74,83	74,78	75,95	78,50	80,64	81,03	80,57	77,31
2009	80,07	79,48	78,96	78,48	78,06	77,50	77,40	78,02	79,96	81,98	82,43	82,09	79,54
2010	81,44	80,86	80,33	79,74	79,22	78,78	78,59	78,63	80,36	83,48	84,49	84,14	80,84
2011	83,72	83,22	82,80	82,28	81,78	81,35	81,15	81,36	82,10	83,29	83,62	83,16	82,49
2012	82,58	81,94	81,38	80,76	80,29	79,84	79,88	80,38	80,85	82,12	82,48	82,09	81,22
Average	78,56	78,18	77,66	77,13	76,76	76,48	76,39	76,78	78,07	79,63	79,85	79,53	77,92

Table C.4: Monthly averaged water levels (in meters) at the Volta Lake. Source: Volta River Authority.

C.3. GENERATED ENERGY BY THE AKOSOMBO DAM

Table C.5 shows the monthly averaged generated power by the Akosombo Dam in GWh.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
1980	14.4	14.5	14.6	14.5	14.4	14.6	14.5	14.3	14.6	13.6	14.5	14.5	**14.4**
1981	14.5	14.8	14.7	14.7	14.5	14.7	14.5	14.4	14.6	14.2	13.0	12.3	**14.2**
1982	11.9	12.1	12.0	11.9	11.7	11.5	11.5	11.7	11.6	11.3	8.7	8.7	**11.2**
1983	8.4	7.6	7.4	7.3	7.0	5.3	4.4	4.4	4.6	4.8	4.5	3.3	**5.8**
1984	3.2	11.1	11.3	10.8	10.9	11.0	10.6	10.4	10.2	10.0	10.5	10.5	**10.0**
1985	5.6	6.0	6.1	6.1	7.0	7.8	6.7	6.7	5.4	6.9	8.4	8.3	**6.7**
1986	8.7	8.8	8.8	8.8	9.3	9.9	10.9	11.0	11.1	11.3	10.9	10.2	**10.0**
1987	10.4	11.1	11.3	10.8	10.8	11.0	10.7	10.4	10.2	10.0	10.5	10.5	**10.6**
1988	10.5	11.1	11.1	11.6	11.8	11.8	10.8	10.4	9.3	10.0	11.2	11.5	**10.9**
1989	11.6	12.0	12.2	12.6	12.5	12.3	12.3	11.6	10.4	11.0	12.4	13.2	**12.0**
1990	13.5	13.4	13.5	13.0	13.1	13.2	12.6	12.9	13.1	13.5	13.2	13.7	**13.2**
1991	14.1	14.5	14.3	14.1	14.2	13.9	13.5	13.2	13.7	13.9	14.9	15.0	**14.1**
1992	15.2	15.6	15.9	15.9	15.3	15.5	15.3	14.8	14.7	14.7	14.9	15.0	**15.2**
1993	14.1	14.5	14.3	14.1	14.1	13.9	13.5	13.2	13.7	13.9	14.9	15.0	**14.1**
1994	16.2	16.2	15.7	15.1	14.5	13.8	13.6	12.5	12.2	11.1	11.9	14.2	**13.9**
1995	14.3	14.6	14.2	13.4	13.3	13.1	13.1	13.4	13.8	14.1	15.2	15.2	**14.0**
1996	15.6	15.8	15.7	15.1	14.9	14.4	14.6	14.6	14.3	14.5	15.9	15.6	**15.1**
1997	16.6	16.7	15.8	15.8	15.2	15.1	14.9	14.6	15.4	16.0	16.2	15.5	**15.6**
Average	**12.1**	**12.8**	**12.7**	**12.5**	**12.5**	**12.4**	**12.1**	**11.9**	**11.8**	**11.9**	**12.3**	**12.3**	**12.3**

Table C.5: Monthly averaged generated energy by the Akosombo Dam in GWh. Source: Volta River Authority.

C.4. KPONG TAIL WATER LEVELS

Table C.6 shows water levels in the Volta River just downstream of the Kpong Dam.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
1996	3,61	3,84	3,79	3,70	3,63	3,55	3,76	3,74	3,59	3,49	3,71	3,81	3,69
1997	3,92	3,98	3,90	3,98	3,98	4,05	3,99	3,94	3,93	3,97	4,00	3,86	3,96
1998	3,50	3,18	2,62	2,34	2,26	2,25	2,18	2,42	2,71	3,20	3,57	3,77	2,83
1999	3,84	3,41	3,32	3,30	3,27	3,59	3,75	3,77	3,50	3,72	3,91	3,94	3,61
2000	3,90	3,84	3,84	3,93	3,93	4,01	3,95	4,41	4,56	4,47	4,58	4,50	4,16
2001	4,49	4,26	4,18	4,22	3,90	3,69	3,74	3,89	3,96	3,91	3,82	3,31	3,95
2002	3,35	3,49	3,11	3,10	3,14	3,21	3,28	3,34	3,43	3,18	3,20	3,19	3,25
2003	2,80	3,06	3,00	2,93	2,86	2,76	2,99	3,27	3,32	3,31	2,90	3,09	3,03
2004	3,02	3,31	3,36	3,22	3,22	3,40	3,47	3,33	3,11	3,21	3,34	3,46	3,29
2005	3,41	3,58	3,27	3,44	3,51	3,49	3,18	3,13	3,33	3,31	3,54	3,63	3,40
2006	3,74	3,55	3,63	3,46	3,66	3,59	3,47	3,31	2,79	2,30	2,46	2,70	3,22
2007	2,87	3,00	2,22	1,74	1,73	1,76	1,64	1,61	1,68	2,24	2,62	2,24	2,11
2008	2,38	2,80	3,21	3,16	3,20	3,21	2,98	3,25	3,32	3,60	3,64	3,40	3,18
2009	3,12	3,35	3,23	3,38	3,54	3,63	3,45	3,25	3,32	3,60	3,64	3,40	3,41
2010	3,80	3,52	3,55	3,58	3,65	3,32	3,31	3,12	2,82	3,40	4,98	3,08	3,51
2011	2,93	3,16	3,30	3,37	3,49	3,50	3,41	3,49	3,66	3,59	3,41	3,35	3,39
2012	3,62	3,76	3,68	3,71	3,78	3,74	3,60	3,42	3,75	3,64	3,82	3,44	3,66
Average	3,43	3,48	3,37	3,33	3,34	3,34	3,30	3,33	3,34	3,42	3,60	3,42	3,39

Table C.6: Kpong Dam tail water levels between 1996 and 2012. Source: Volta River Authority.

C.5. BED SLOPE AND MANNING COEFFICIENTS

For the bed slope, the value $0.002 m/m$ is used. For the Manning's n values, $0.006 m^{-1/3}$ and $0.033 m^{-1/3}$ are used to define the roughness of the banks and the main channel of the river respectively. Those values were also used by Logah et al. (2017) for setting up a hydrodynamic model of the Lower Volta River [15].

C.6. BED LOAD TRANSPORT

Fig C.1 shows the distribution of the bedload fraction. These data were also collected by Logah et al. (2017) [15].

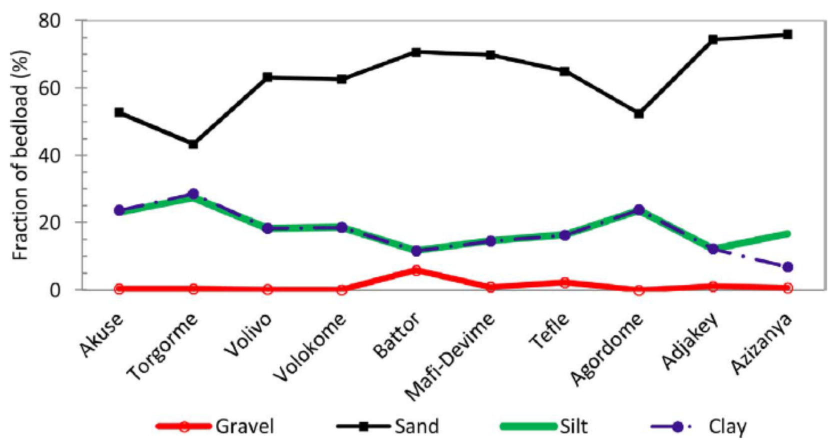

Figure C.1: Bedload distribution and soil type along the Lower Volta Riter. From Logah et al. (2017) [15].

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